VIRGINIA TECH DUCK POND RETROFIT FOR IMPROVED WATER QUALITY IN STROUBLES CREEK

by

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David F. Kibler, Chair

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(ABSTRACT)

Stroubles Creek is registered on Virginia's 303(d) list of impaired waters for both benthic and fecal coliform impairments. The upper reach of the creek's watershed drains into two ponds on the Virginia Tech campus. The area draining to the ponds, approximately 715 acres, encompasses most of the Town of Blacksburg and the Virginia Tech campus. Below the ponds, the creek's watershed is primarily forested and agricultural, with some areas of residential development.

In order to improve water quality downstream, the two ponds will be converted to a water quality facility by redirecting all flow from the northern branch of Stroubles Creek into the upper, smaller pond, which then flows into the larger pond below. With flow into the upper pond increasing dramatically, the dam between the two ponds and associated overflow structures were evaluated and redesigned to protect the dam from overtopping and possible washout. In addition, concrete weirs were designed and will be constructed on both branches of Stroubles Creek above the ponds for future installation of flow and water quality monitoring equipment. Above the ponds, the banks along both branches of the creek have become severely eroded. Interlocking concrete block armoring was designed for the stream banks to reduce erosion and protect the trees growing along the creek.

This project was jointly funded by Virginia Tech and a grant from the Virginia Department of Conservation and Recreation Water Quality Improvement Fund. Construction will be performed by the Capital Design department of Virginia Tech.

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CHAPTER 1
PROJECT BACKGROUND

1.1 PROJECT NEED AND BASIS

The Clean Water Act is, in fact, not a single piece of legislation, but an amendment to a series of legislative acts passed by Congress between 1956 and 1972. The 1977 amendments to the Federal Water Pollution Control Act establish guidelines for the states to follow with the goal of "restor[ing] and maintain[ing] the chemical, physical and biological integrity of the Nation's waters" (Clean Water Act 1987, sec. 101(a)). Instead of having such a monumental task managed centrally at the Environmental Protection Agency (EPA), the Clean Water Act gives the responsibility of improving water quality to the state governments. Each state is required to develop a "comprehensive program for preventing, reducing, or eliminating the pollution of the navigable waters and ground waters" (Clean Water Act 1987, sec. 102(a)).

In addition to eliminating point source pollution, each state is also required to develop its own standards for water quality and compare all surface waters against those standards. The aim of these water quality standards is to ensure that the waters of the United States are suitable for "the protection and propagation of shellfish, fish, and wildlife" and "to allow recreational activities in and on the water" (Clean Water Act 1987, sec. 101(a)(2)). For each body of water that does not meet the state water standards, e.g. is "impaired", a plan must be developed that identifies how the water is impaired, what pollutants are causing the impairment(s), where the pollutants are coming from, and that proposes a remediation plan to meet the standards. This list of impaired water is often referred to as the "303(d) list," alluding to the section of the Clean Water Act where it is mandated.

Each remediation plan must include a determination of the total maximum daily load (TMDL) for each of the pollutants responsible for the water's impairment(s). The TMDL for a particular pollutant is the maximum in-stream amount that a given body of water may contain without causing impairment. One important aspect of TMDL development is that the target pollutant must be measurable, e.g. quantifiable. Some of

the "conventional" pollutants listed in the Clean Water Act include biological oxygen demand (BOD), suspended solids, fecal coliforms, and pH (Clean Water Act 1987, sec. 304(a)(4)). Other pollutants include pesticides, heavy metals, polychlorinated biphenyls (PCBs), and nutrients, particularly nitrogen and phosphorus. Temperature may also be considered a pollutant, but is not listed as a conventional pollutant (Clean Water Act 1987, sec. 304(a)(4)). Since TMDL pollutants must be quantifiable, "protection and propagation of wildlife" cannot itself be used as a TMDL. In order to quantify the health of wildlife in a body of water, samples of benthic (bottom-dwelling) macroinvertebrates are often taken and counted. Using one of several index methods, the overall biological health of a stream can then be extrapolated from the number and diversity of macroinvertebrates present. This standard is often referred to as the "benthic" standard.

Stroubles Creek was originally listed on Virginia's 303(d) list in 1998 for violation of the benthic standard; in 2002, it was listed for violation of the fecal coliform standard in several reaches below the Duck Pond. Studies are currently in progress at Virginia Tech to determine the source of the pollutants causing these violations and the TMDLs for the pollutants. Once these studies are complete, a plan for mitigating the pollutants and meeting water quality standards must be developed. The Duck Pond Retrofit project was funded by the Virginia Department of Conservation and Recreation (DCR) as a first step toward remediation of Stroubles Creek.

1.2 PROJECT GOALS

The primary goal of the Duck Pond Retrofit project is to create a two-pond water quality facility from the upper and lower ponds with the goal of improving water quality in Stroubles Creek downstream of the ponds. The water quality facility will be created simply by removing a bypass channel that parallels Duck Pond Drive. Removing this bypass channel will force all flow from the northern branch into the upper pond, thereby capturing a greater portion of "first flush" pollutants in the upper pond. Future phases of the overall project will include installation of flow and water quality monitoring equipment to evaluate the performance of the two-pond facility.

The specific goals contained in this project report are:

1) Create a hydrologic model of the area draining to the upper pond.
2) Design modifications to the bypass channel so that flow is redirected into the upper pond.
3) Design modifications to the upper pond dam to ensure its stability during large rainfall events.
4) Design and test two weirs for future installation of flow and water quality monitoring equipment.
5) Design measures to stabilize the banks of the northern and southern branches of Stroubles Creek between West Campus Drive and the Duck Pond.
6) Make several modifications to the Virginia Tech version of the Penn State Urban Hydrology Model (VTPSUHM) computer program.

1.3 A BRIEF HISTORY OF THE DUCK POND

Virginia Tech was originally founded as the Virginia Agricultural and Mechanical College in 1872. Soon thereafter, the upper pond (figure 1.1) was built on the northern branch of Stroubles Creek, near a farmhouse that was part of a teaching farm on campus. That farmhouse, now known as Solitude, is still used by the University and is considered a historic structure. The pond was used as a source of ice until 1898 when a refrigeration plant was constructed on campus. There is no evidence of what engineering calculations were used in the design or construction of the upper pond.

The lower pond was designed by a Virginia Tech civil engineering student in 1913 for his undergraduate thesis (Fowle 1913). The pond was finally constructed in 1937, during a wave of campus construction supported by President Roosevelt's New Deal programs. In addition to the pond and other projects, eight major campus buildings including Burruss, Squires, and Eggleston Halls were built during the period between 1936 and 1940 (Wallenstein and Kennelly 2001). During this same period, a section of the southern branch of Stroubles Creek was enclosed and filled over to create the Drillfield.

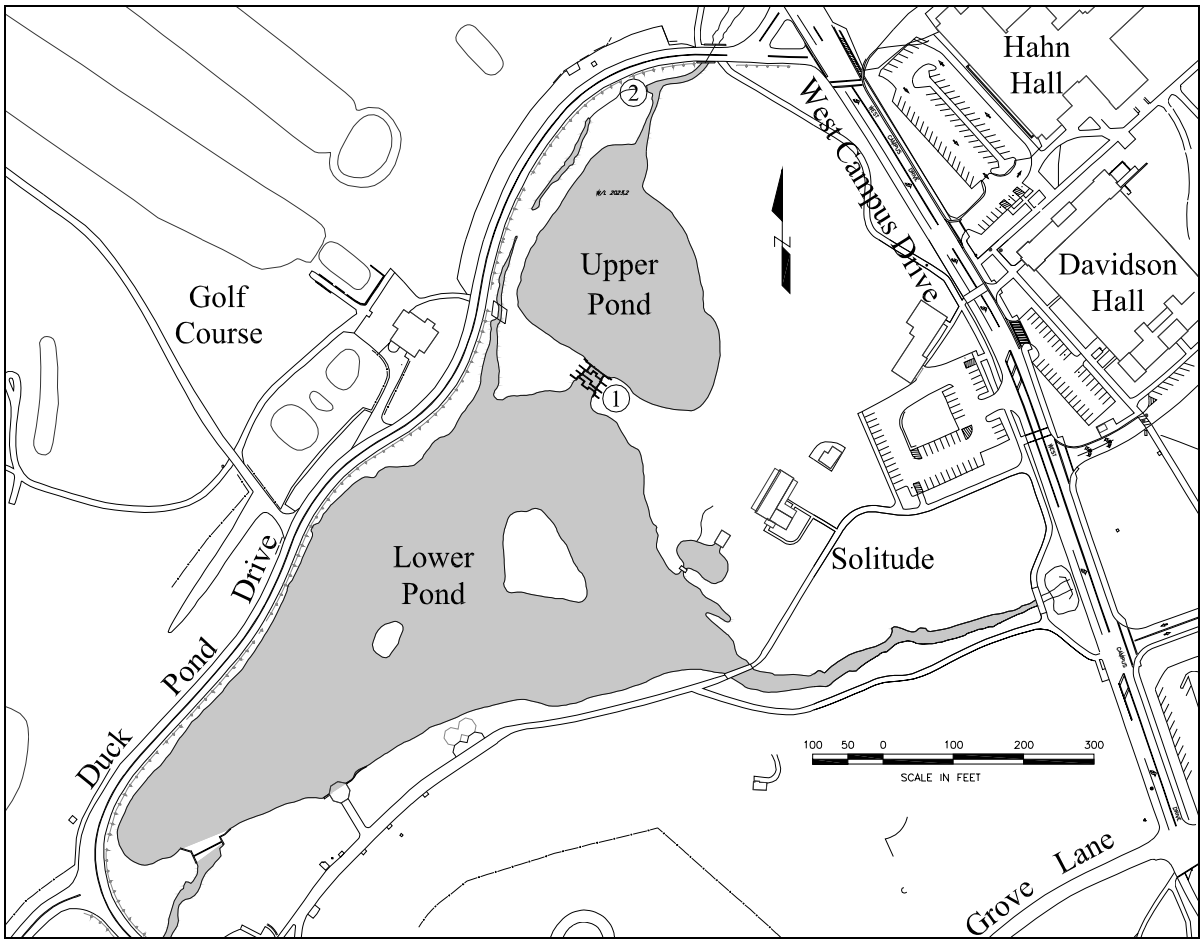

Figure 1.1 – Location of upper Duck Pond on the Virginia Tech campus.

The concrete "staircase" structure between the upper and lower ponds (location #1 in figure 1.1) was built in the late 1980's. It was designed by the University landscape architect, and was intended to be a more attractive and more durable alternative to the rock-lined earthen channel that had previously connected the ponds. Although this structure was not engineered, it functions both as a low-flow channel connecting the ponds and as a mechanism for aeration. The structure has held up very well since its construction, as has the dam around it. Only slight erosion of the embankment has occurred immediately adjacent to the structure.

The staircase structure at the inlet of the bypass channel (location #2 in figure 1.1) was designed and built concurrently with the larger staircase. This structure, intended to be an improved inlet that would only allow flow into the bypass channel during rainfall events, was also designed by the University landscape architect and was not engineered.

Unfortunately, it has not fared as well as the larger staircase. Large storms over the last 14 years have severely eroded the banks on either side of the smaller staircase structure, and have undermined its slab stone sides. This erosion may also be responsible for settling of the structure, which has lowered its invert enough that the structure carries significant flow even during low flow in the stream. By visual estimation, the bypass channel appears to carry approximately a third of the flow in the northern branch of Stroubles Creek.

Both the upper and lower ponds have little available head and storage above the normal pool elevation. The normal pool elevation of the upper pond is approximately 1.1 feet below the top of the dam between the ponds, and the lower pond normal pool is approximately 2.0 feet below the top of the lower dam. The maximum available storage above the upper pond normal pool is 2.1 acre-feet at elevation 2024.5'. If the dam were raised to elevation 2025.5', the available storage would be doubled. However, the finished floor of Solitude is at elevation 2028.6', and the existing grade outside the building is at approximately 2026.5' at its lowest point. Protection of this historic building must be taken into account in the design of any changes to the upper pond.

1.4 THE NEED FOR MODELS

As mentioned in the previous section, removing the bypass channel will greatly increase the flows into the upper pond. With little head and storage capacity available in the upper pond, chances of the dam being overtopped will be much greater. Overtopping must be prevented, or the dam must be protected against its erosive effects.

In order to design modifications to the dam, the quantity and distribution of stormwater entering the pond must be determined, e.g. hydrographs must be developed. There are no gaging stations on Stroubles Creek above the Duck Pond, so there are no observed flow data available for use on this project. Therefore, stormwater models must be developed to obtain the needed hydrographs. An in-depth discussion of model development for this project is contained in Chapter 2.

1.5 PROJECT HISTORY

In response to the Clean Water Act, the Commonwealth of Virginia passed the Water Quality Improvement Act in 1997. The stated goal of this Act is "to restore and improve the quality of state waters and to protect them from impairment and destruction" (VA DCR 2002). As part of the Water Quality Improvement Act, the Water Quality Improvement Fund (WQIF) was also created to provide funding for projects that improve water quality in the state. The fund is intended primarily for projects that seek to reduce nitrogen and phosphorus loading in the Chesapeake Bay, but any project with the goal of improving water quality may receive funding. Private individuals and organizations, as well as state agencies, may apply for funding from the WQIF on a cost-sharing basis.

During the fall of 2000, Dr. David Kibler of the Civil and Environmental Engineering department and Dr. Tamim Younos of the Virginia Water Resources Research Center collaborated with the Capital Design and Construction department of Virginia Tech to develop a grant proposal for this project, with the aim of reducing nonpoint source pollution in Stroubles Creek. The grant was approved in 2001, and funds were made available in the spring of 2002. Unfortunately, due to state budget cutbacks, the WQIF did not receive any funding during the 2002 fiscal year, so work on future phases of this project may be delayed.

CHAPTER 2
HYDROLOGIC MODEL DEVELOPMENT

2.1 DATA COLLECTION

In 1994, the Town of Blacksburg and Virginia Tech jointly hired a Roanoke, VA engineering firm, Hayes, Seay, Mattern & Mattern (HSMM), to create a comprehensive stormwater model of Blacksburg and the Virginia Tech campus. This model was created in XP-SWMM, a proprietary version of the EPA's Storm Water Management Model (SWMM). The model was based on aerial topographic data taken in 1994 and field surveys performed by HSMM, and was completed in the fall of 1995. Unfortunately, neither the Town nor the University has been able to make the model fully functional for engineering purposes.

Although the model had never been operational, it was hoped that some useful data could be extracted from the input data files and used in EPA-SWMM. The University's copy of the model was stored on 74 3.5" floppy disks. Extracting the data files took several hours and the extracted files contained many errors, likely due to the tendency of floppy disks to be unstable for long-term data storage. The disks were divided into several subcategories: Existing (100, 50, 25, 10, and 2-year), Future (100, 50, 25, 10, and 2-year), University (100, 10, and 2-year), University with Vet Pond (100, 10, and 2-year), CEC Bypass (100 and 10-year), Future Reg. Pond (100 and 10-year), and Floodway Boundary Maps. There was no documentation included on the data disks, so it was unclear what these categories referred to, what information was included in them, or why certain return periods were included in some categories and not others.

The format of the data files was similar to that of EPA-SWMM data files. As shown in figure 2.1, the data are entered into a text file on individual lines, or "cards," each with an alphanumeric identifier at the start of the line. Each card contains information about rainfall, watershed subareas, pipes, or other pertinent data. Both EPA-SWMM and XP-SWMM have the same basic functions; one component of the program uses rainfall input data to calculate the runoff from each subarea, then another component of the program routes this runoff through the pipe and channel network in the watershed.

In EPA-SWMM, the runoff component of the program is simply called the Runoff Block, and the transport component is referred to as the Extended Transport Block, or EXTRAN.

```
*  KTYPE KINC KPRINT KTHIS KTIME KPREP NHISTO THISTO TZRAIN
E1   0    10    0      0     1     0     49    0.5      0
*
*  10-YEAR STORM FROM VTPSUHM (RAINFALL RATE, TYPE II SCS)
E3 0.000,0.052,0.054,0.056,0.058,0.060,0.063,0.065,0.068,0.072,
E3 0.075,0.080,0.085,0.090,0.097,0.104,0.114,0.125,0.140,0.160,
E3 0.187,0.229,0.302,0.472,3.723,0.704,0.365,0.260,0.206,0.172,
E3 0.149,0.132,0.119,0.109,0.100,0.093,0.087,0.082,0.078,0.074,
E3 0.070,0.067,0.064,0.061,0.059,0.057,0.055,0.053,0.051
*
*  SUBWATERSHED DATA
*  JK  NAMEW  NGTO WIDTH AREA  %IMP  SLP   IMPN  PERVN  IDS   PDS   SUCT  HYDCON SMDMAX
H1  1    1    101  2350  39.7   40  0.074  0.014 0.20  0.016  0.25  10.75  0.08  0.4
H1  1    2    201  2945  71.0   22  0.089  0.014 0.20  0.016  0.25  10.75  0.08  0.4
H1  1    3    301  2355  50.0   14  0.083  0.014 0.20  0.016  0.25  10.75  0.08  0.4
H1  1    4    601  2915  60.9   13  0.127  0.014 0.20  0.016  0.25  10.75  0.08  0.4
```

Figure 2.1 – Sample input data for EPA-SWMM Runoff Block.

Input files for the Runoff Block contain general information about units, equations to be used for infiltration and evaporation, the number of rainfall gages within the watershed, and print options. More importantly, the Runoff input files contain information about snowfall, rainfall (in the form of hyetographs), groundwater, water quality, and the surface characteristics of each subwatershed. These subwatershed data are contained in the H1 cards as shown in figure 2.1, and include the area of each subwatershed in acres, which inlet the subarea is draining to, the percent impervious area of the subwatershed, average slope in feet/feet, the Manning's "n" for both impervious and pervious surfaces, and infiltration parameters.

The H1 cards are where the first major differences appeared between the XP-SWMM data files and the EPA-SWMM format. The XP-SWMM format had several extra columns on the H1 cards, and no correlation could be made between the XP-SWMM and the EPA-SWMM formats. Also, the XP-SWMM model uses a second routing module, Transport, in addition to EXTRAN. No documentation could be found on Transport or its data structure, so it is still unknown what data are contained in that section of the XP-SWMM input files.

The inclusion of Transport in the XP-SWMM model also eliminated the possibility of using a conversion program to convert the XP-SWMM files to EPA-SWMM format. Two conversion programs were tried, and both failed to convert the files. Even with the input files broken apart into separate Runoff, Transport and EXTRAN files, the conversion routines still failed to convert them. The only useful information that could be extracted from the XP-SWMM data files were the hyetographs used by the model.

As mentioned above, a portion of the disks contained floodway boundary maps. These AutoCAD files contained 2' contour-interval mapping of the Town, including the location and sizes of storm sewers and culverts. This AutoCAD mapping turned out to be useful as a source of several key pieces of information, as discussed in the next section.

2.2 DEFINITION OF SUBAREAS

There are some basic data such as area, average slope, and surface cover that must be determined for a watershed that is modeled in a spatially distributed model such as EPA-SWMM. The watershed must also be divided into smaller subwatersheds in order to increase the accuracy of the model. The AutoCAD mapping included in the HSMM data files was an essential element in both of these procedures. Figure 2.2 shows the outline of the upper pond watershed overlaid with the Blacksburg mapping units that the watershed covers.

The 715-acre watershed was then divided into 12 subwatersheds ranging from 40 to 95 acres. It was first divided into fifteen subwatersheds, but some were too small to be used. EPA-SWMM has a tendency to become numerically unstable when the range of pipe lengths exceeds an order of magnitude (i.e. from 100' to 1000'). Since pipe length in the model is controlled by subwatershed size, the smallest subwatersheds were combined with neighboring areas resulting in the final twelve-subwatershed arrangement. Since the effect of the number and size of subwatersheds on the model output is highly variable, a 34-subwatershed layout was also created, with subwatersheds ranging from 9 to 55 acres, for comparison. Both layouts are shown in figure 2.3. The 12-subwatershed gave more conservative results during initial testing of the two layouts under 2-year conditions (figure 2.3b), so the 12-subwatershed layout was used for final modeling.

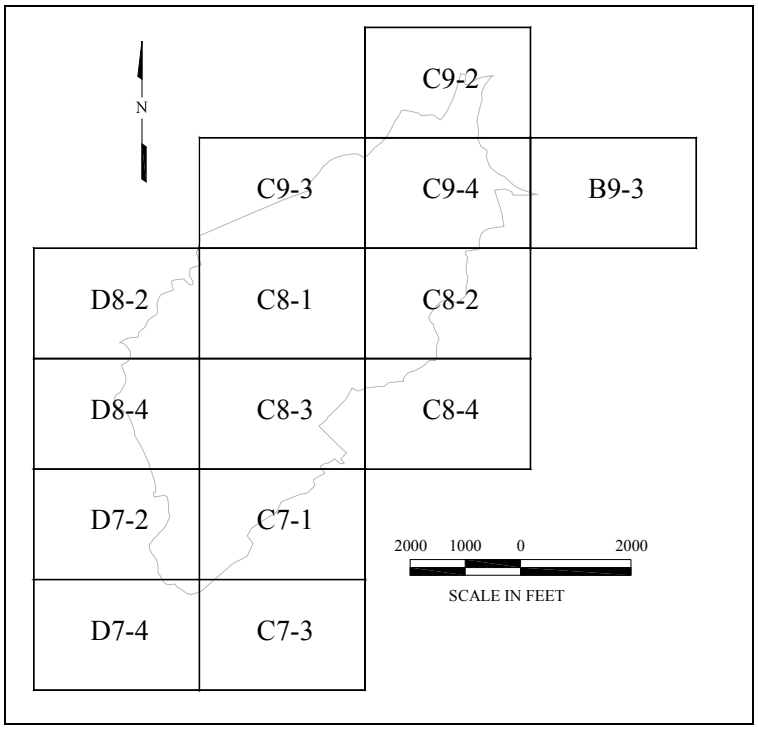

Figure 2.2 – Extent of Town of Blacksburg mapping units covering upper pond drainage basin.

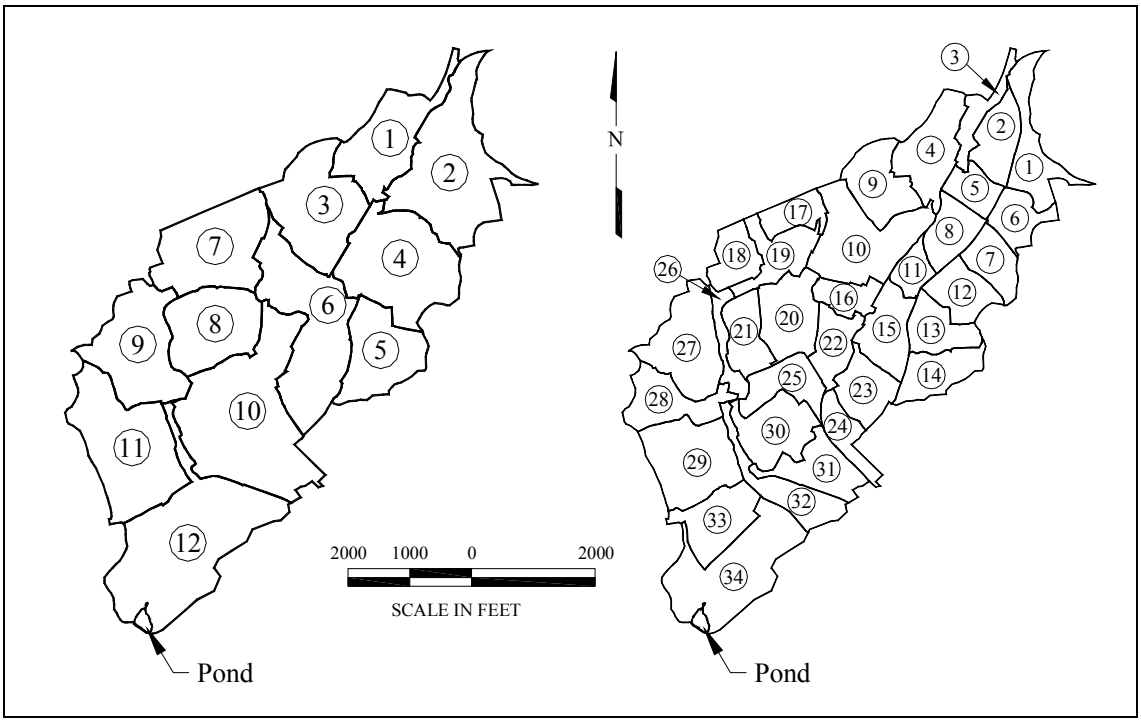

Figure 2.3a – 12- and 34-subwatershed layouts.

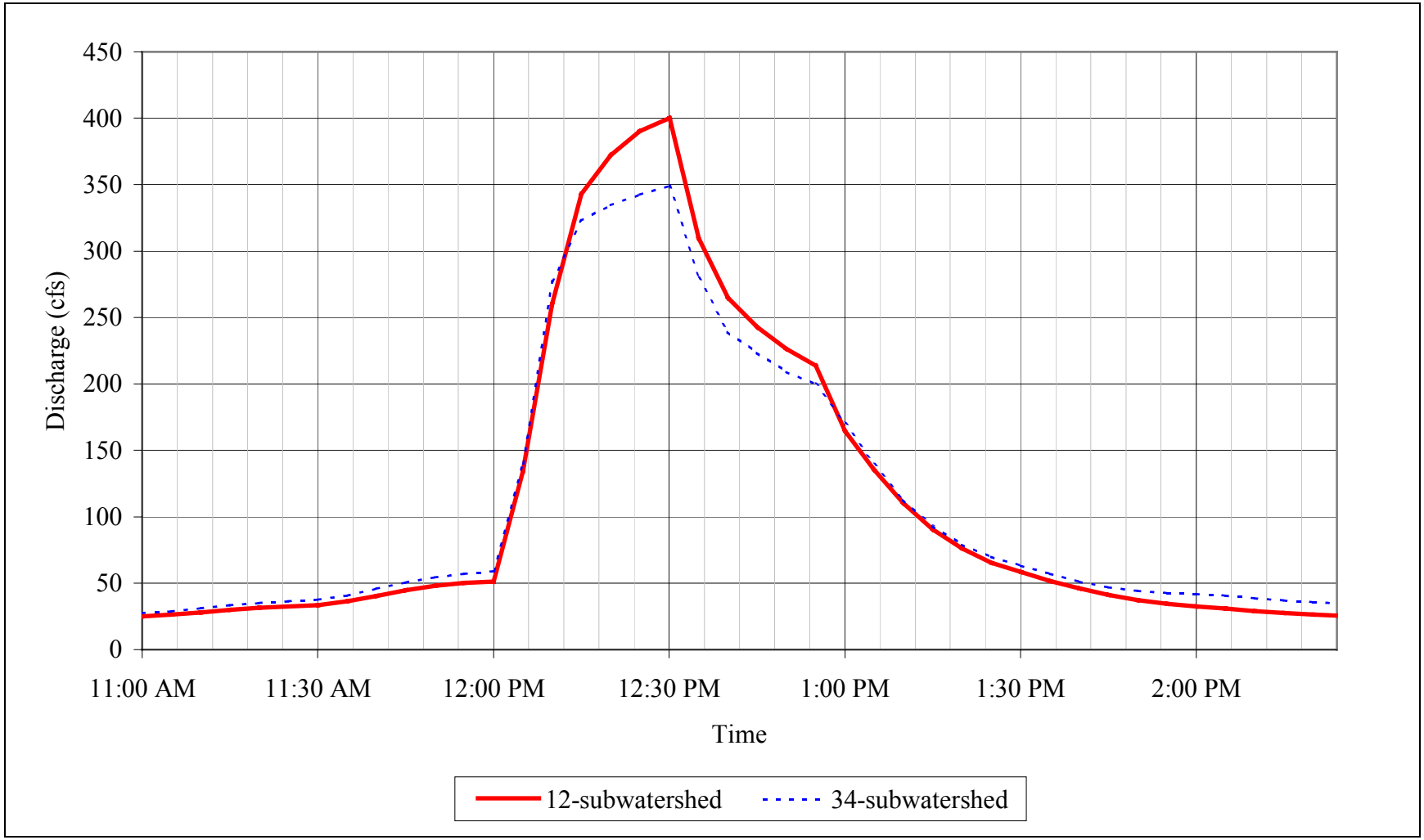

Figure 2.3b – 12-and 34-subwatershed preliminary hydrographs for the 2-year storm.

Major changes in drainage patterns such as junctions of pipes or channels, large culverts, or road embankments were used as beginning points for delineating subwatersheds. Once the outlet of a subwatershed was fixed, its boundaries could be determined from the topographic contours and the pipe network in the drawing. The drainage area of each subwatershed was easily determined from closed polylines drawn around each subwatershed boundary. Average slope and impervious fraction, however, were more difficult to obtain.

Using Land Development Desktop (LDD) and the contours in the drawing, a triangular irregular network (TIN) was created for each subwatershed. These three-dimensional surfaces not only allow for easy visualization of the land surface, but can also be used to calculate earthwork volumes, create land surface profiles, or generate cross-sectional views along an alignment. LDD also calculates an average slope of all the triangular surfaces in the TIN as part of its calculations. This slope is equivalent to the average ground slope within the subwatershed.

Due to the large area covered by the watershed (715 acres), determining the impervious fraction within each watershed by summing the individual areas of AutoCAD objects such as polylines would have been extremely time consuming. In order to lessen the amount of work involved, a scheme using average areas was utilized. Figure 2.4 shows subwatershed #7, which includes a large portion of the Terrace View apartment complex. This subwatershed is bounded on the north by Patrick Henry Drive, on the east by Progress Street, on the south by Broce Drive, and on the west by Tom's Creek Road.

In order to determine the amount of impervious area within this subwatershed, first the number of apartment buildings in the area was counted. Some buildings are obviously twice the size of the average building in the subarea, so these were counted as two buildings. The number of apartment buildings was then multiplied by an average footprint, determined by averaging five or six representative buildings. A similar method was used for single-family houses. Second, the total length of roadway within the subwatershed was calculated by drawing polylines along the road centerlines and summing the lengths of the polylines. This total length of roadway and parking lot was then multiplied by an average road width. This width was calculated by averaging the typical parking lot and typical roadway widths, weighted by length as:

$$w_{avg} = \frac{w_r l_r + w_p l_p}{l_r + l_p} \qquad [2\text{-}1]$$

where w_{avg} is the average width of pavement, w_r and w_p are the typical widths of roadways and parking lots, respectively, and l_r and l_p are the total lengths of roadways and parking lots, respectively. Any large impervious areas that were not included in either of the above methods were added separately. Table 2.1 shows the impervious area calculations for all twelve subareas.

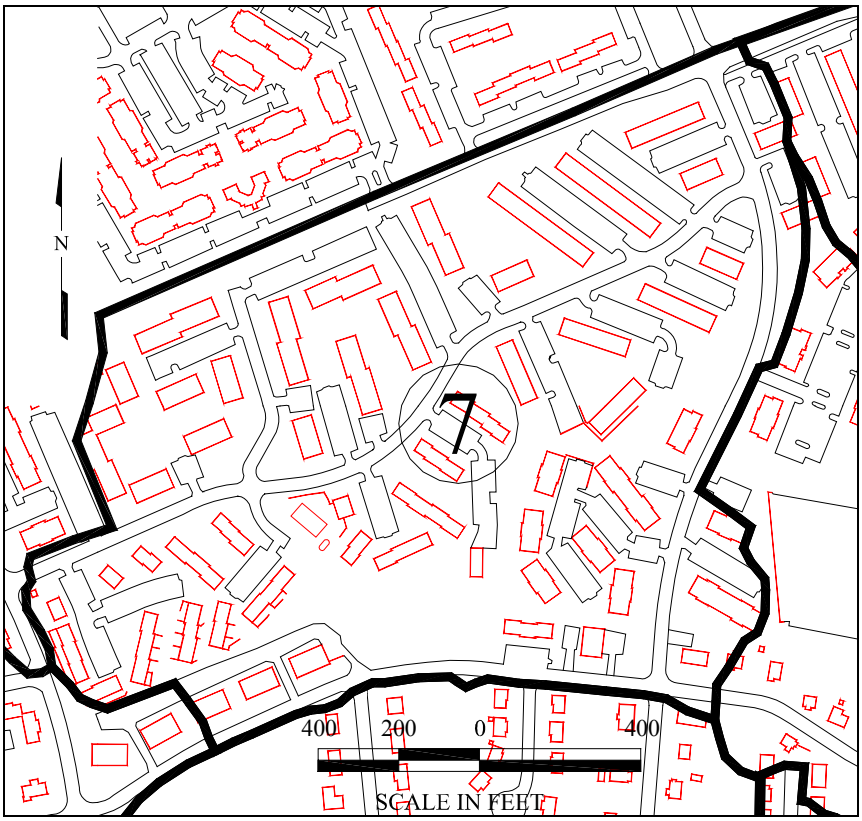

Figure 2.4 – Impervious areas within subwatershed #7.

The impervious fractions used in the EPA-SWMM model represent the current level of development within the watershed. Future development was not accounted for since the watershed is already highly developed, with a few open areas in the upper reaches of the watershed. If these open areas were developed, their effect on flow to the upper pond would be minimal due to their location in the watershed.

Table 2.1 – Impervious area calculations.

Area #	Road length ft	Road width ft	# Houses	House area sf	# Apt Bldgs	Apt bldg area sf	Other sf	Total Imp. Area sf	Total Imp. Area ac	% Imperv.
1	6815	45	1	4000	5	4380	355,340	687,915	15.79	39.8%
2	9125	32	80	2400	18	6275	82,650	679,600	15.60	22.0%
3	1160	50	7	2600	14	5270	152,890	302,870	6.95	13.9%
4	5095	30	37	1400	2	5000	123,380	338,030	7.76	12.7%
5	7080	35	44	2675	0	0	0	365,500	8.39	23.8%
6	7515	25	95	1000	22	4300	447,955	825,430	18.95	28.8%
7	15395	35	0	0	63	5350	0	875,875	20.11	39.6%
8	6000	32	83	1450	0	0	0	312,350	7.17	18.0%
9	8790	35	90	1800	0	0	0	469,650	10.78	22.0%
10	18620	50	132	1650	45	4500	388,315	1,739,615	39.94	42.3%
11	9810	32	111	2050	0	0	0	541,470	12.43	19.5%
12	0	0	0	0	0	0	2,795,300	2,795,300	64.17	67.5%

2.3 INFILTRATION MODEL SELECTION

EPA-SWMM allows the user to select one of two infiltration models: Horton or Green-Ampt. The Horton model was developed in the late 1930's and is a purely empirical model of the form:

$$f = f_c + (f_o - f_c)e^{-Kt} \qquad [2\text{-}2]$$

where f is the infiltration rate (in/hr) at time t (sec), f_c is the minimum, or asymptotic infiltration rate (in/hr), f_o is the initial infiltration rate (in/hr), and K is the decay rate (1/sec) (Horton 1939, 1940). The decay rate and initial infiltration rate are highly dependent on the initial moisture content of the soil, and the minimum infiltration rate is generally less than or equal to the saturated hydraulic conductivity of the soil (Haan et al. 1982). For greatest accuracy, all parameters should be determined from experimentation, though general guidelines have been developed over the years for different soil types.

In contrast to the Horton infiltration equation, the Green-Ampt model is based more on the physical processes that take place within the soil. Specifically, it is derived from Darcy's Law. One of the model's basic forms, developed in 1911 by Green and Ampt, is (Green and Ampt 1911):

$$f = K_s + K_s S_{av} M/F \qquad [2\text{-}3]$$

where f is the infiltration rate (in/hr), K_s is hydraulic conductivity (in/hr), M is the difference between initial and final water content $(M = \theta_s - \theta_i)$, S_f is the effective suction at the wetting front (in.), and F is the cumulative infiltration (in.). Later, Mein and Larson modified equation [2-3] by assuming that ponding does not occur immediately after rainfall begins (Mein and Larson 1973). First, they calculated the cumulative infiltration at the time ponding begins, F_p (in.):

$$F_p = \frac{S_{av} M}{R/K_s - 1} \qquad [2\text{-}4]$$

where S_{av} is the average suction at the wetting front (in.) and R is rainfall rate (in/hr). Thus the infiltration rate is:

$$f = R \qquad \text{for } t < t_p \qquad [2\text{-}5a]$$

and

$$f = f_p = K_s + K_s S_{av} M/F \qquad \text{for } t > t_p \qquad [2\text{-}5b]$$

where t_p is the time to ponding (hr).

Because it is based on physical processes and is used successfully in many different models and applications, Green-Ampt was the infiltration model chosen for this project in EPA-SWMM. In addition, calculations could be performed to confirm that Green-Ampt was being applied correctly within EPA-SWMM. Figure 2.5 is a comparison of the infiltration curve obtained from EPA-SWMM and the curve obtained by manually calculating infiltration rates using equations [2-5a] and [2-5b] and a constant rainfall rate of two inches per hour. The time to ponding, decay rate, and minimum infiltration rate all match very closely.

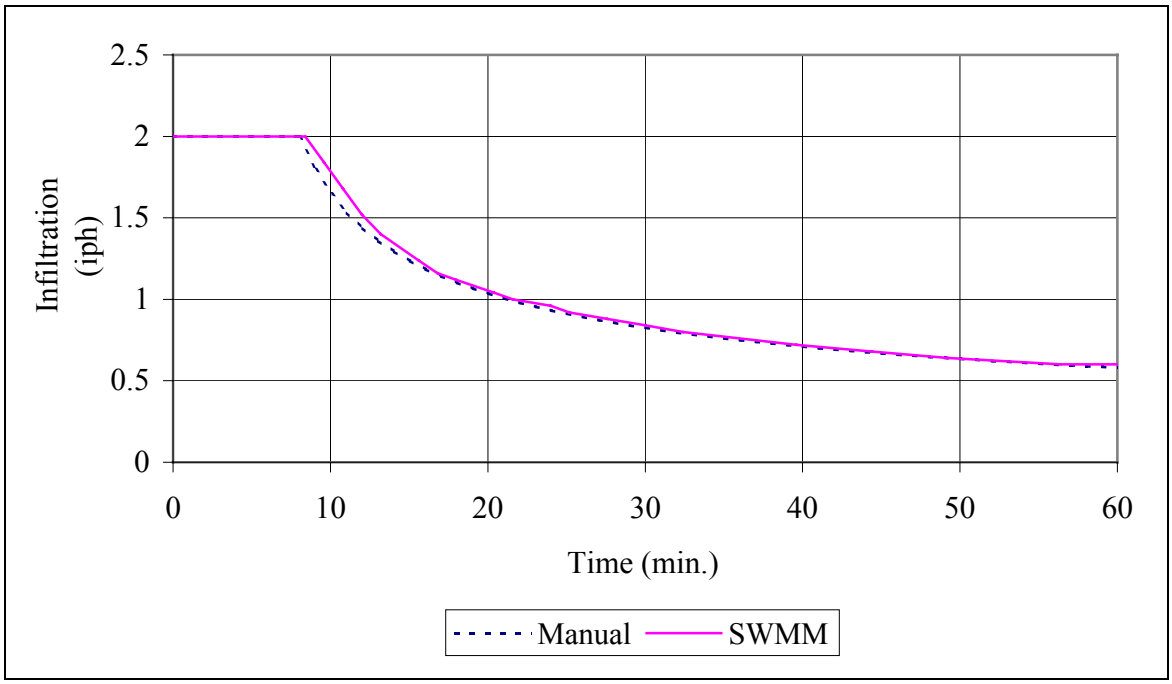

Figure 2.5 – Comparison of Green-Ampt infiltration rates for steady rainfall of 2 iph.

Because Green-Ampt is physically based, it can be difficult to apply. The required inputs for Green-Ampt in EPA-SWMM are average capillary suction (SUCT, in.), saturated hydraulic conductivity (HYDCON, in./hr), and initial moisture deficit (SMDMAX, volume air / volume voids) (Huber and Dickinson 1992). According to the EPA-SWMM user's manual, the model outputs are relatively insensitive to changes in HYDCON and SUCT, but are highly sensitive to the value of SMDMAX chosen. In order to confirm these conclusions, a sensitivity analysis was performed with respect to both HYDCON and SMDMAX. Initial values were chosen based on the dominant soil types within the watershed.

There are seven predominant soil types within the drainage basin of the upper pond according to the Soil Survey of Montgomery County (Creggar et al. 1985). These soils are described in Appendix A. Six of the seven soils generally consist of a surface layer of silt loam ranging from 6 – 12" thick and a subsoil of silt loam, clay loam, and clay ranging from 29 – 64" thick. The subsoil is more likely to control infiltration since it contains more clay, so clay loam was used as an overall soil descriptor in choosing infiltration parameters.

First, various values of SMDMAX were chosen ranging from 0.1 to 0.9 and modeled using HYDCON = 0.08 in/hr and SUCT = 10.75 in. The results for the 2-year storm at the upper pond are shown in figure 2.6. Although the hydrograph tails varied widely in terms of timing, the peak flows and times to peak were relatively close. Values for peak flow ranged from 388 cfs to 425 cfs, a difference of less than 10%. Relative to the differences in peak flows obtained during model comparison (discussed in the next section), this difference is negligible.

Next, values of HYDCON were tested at SMDMAX = 0.1 and SMDMAX = 0.5 (SUCT = 10.75”). These values of SMDMAX were chosen in order to give more conservative peak flows. Since the soils in the subject watershed are highly variable and generally disturbed due to development and urbanization, a wide range of values for HYDCON was tested. Values ranged from 0.008 to 0.8 in/hr, a difference of two orders of magnitude. Results are shown in figures 2.7 and 2.8. Peak flows at SMDMAX = 0.5 ranged from 364 cfs to 424 cfs, a difference of 16%. Peak flows at SMDMAX = 0.1 ranged from 389 cfs to 433 cfs, a difference of 11%. Again, these ranges are negligible relative to those found in the next section. Peak outflow appeared to be relatively insensitive to variations in either SMDMAX or HYDCON.

A value for SMDMAX of 0.4 was chosen for the final model based on the above analysis. However, after further investigation, an initial moisture content of 0.24 would approximate the wilting point for clay loam soils according to Huber and Dickinson (1992). Therefore, a value of 0.1 for SMDMAX would have been more suitable to represent a “moist” watershed with clay loam soils. Revising the 25-year adjusted EPA-SWMM model (section 2.4) revealed that the 2-year peak flow would have increased nearly 10% using SMDMAX = 0.1, although the higher return period peak flows would have increased less than 1% (table 2.2). Thus, the model was not revised to use SMDMAX = 0.1 given the small differences in the larger return period storms.

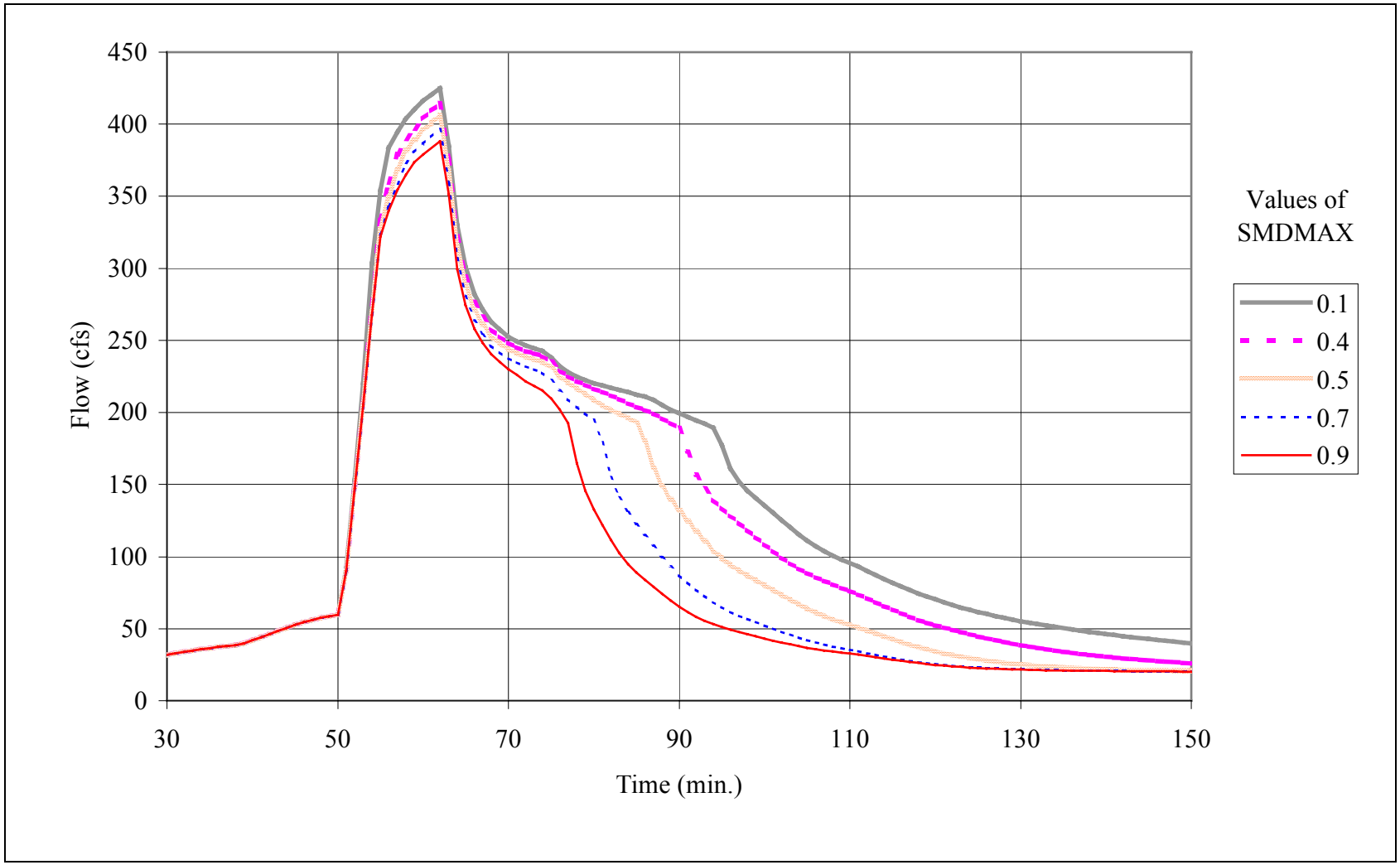

Figure 2.6 – Sensitivity analysis on initial moisture deficit (SMDMAX) for the 2-year storm.

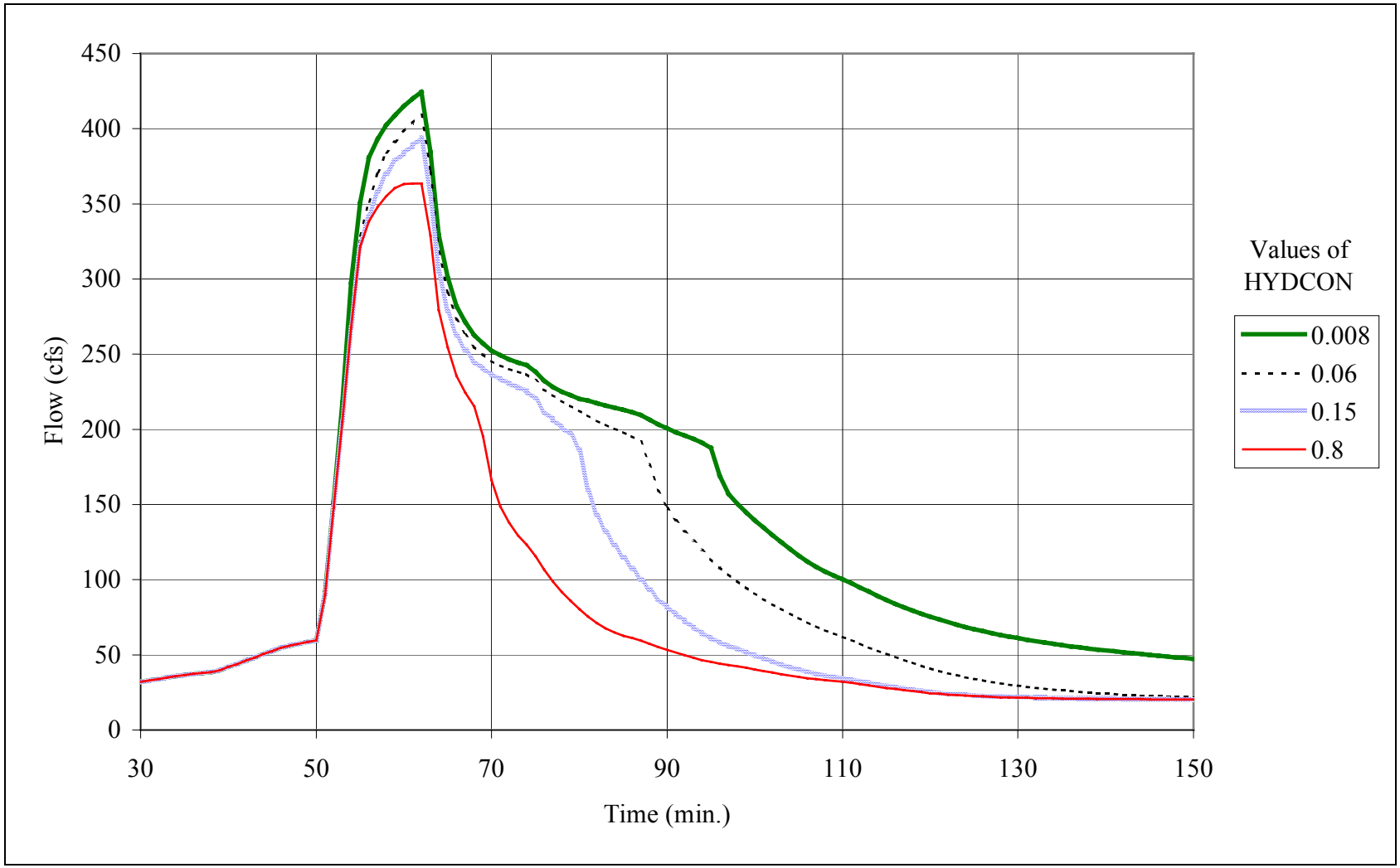

Figure 2.7 – Sensitivity analysis on hydraulic conductivity (HYDCON) at SMDMAX = 0.5 for the 2-year storm.

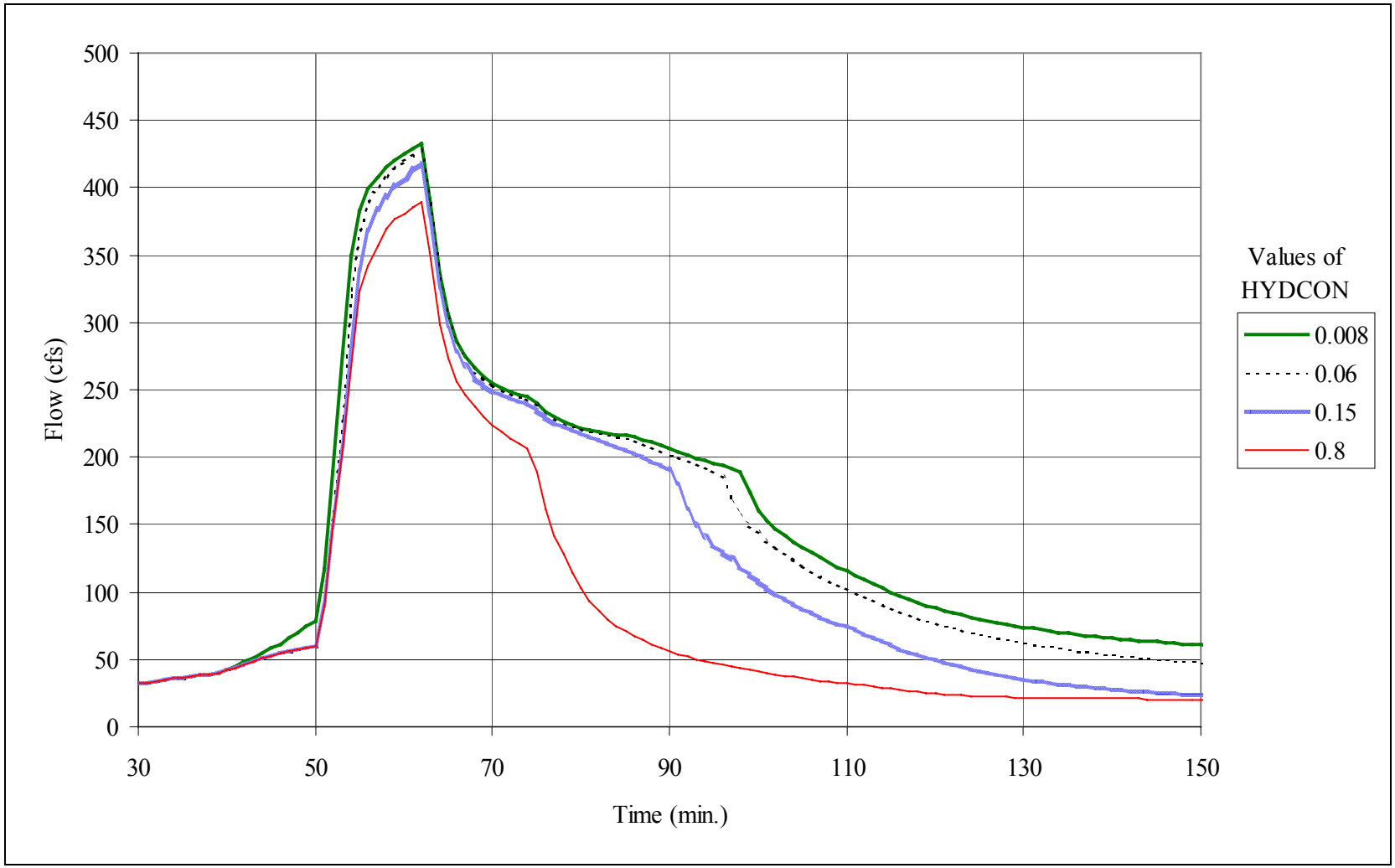

Figure 2.8 – Sensitivity analysis of hydraulic conductivity (HYDCON) at SMDMAX = 0.1 for the 2-year storm.

Table 2.2 – Comparison of peak flows at the upper pond with variable SMDMAX.

Return Period yrs	Peak Outflow		Increase
	SMDMAX = 0.4 cfs	SMDMAX = 0.1 cfs	
2	922	1013	9.9%
10	1376	1389	0.9%
25	1430	1441	0.8%
50	1492	1504	0.8%
100	1544	1556	0.8%

A value for HYDCON of 0.08 in/hr was chosen for the final model. This value is within the range of values for hydrologic group C soils given by Huber and Dickinson (1992), and should provide conservative results for clay loam soils. A value of 10.75" for SUCT was used in all EPA-SWMM models. This value is between those for silt loam and clay loam given by Huber and Dickinson (1992).

2.4 COMPARISON OF MODELS

As mentioned in section 2.2, the 12-subwatershed layout was used to model all storms. However, since there are no gaging stations above the two ponds, there were no observed flow data to compare against the EPA-SWMM model results. Therefore, several "simple" stormwater models were used to generate comparative data. These models were: (1) the USGS basin development factor (BDF) method; (2) the Stankowski ratio; (3) the Stankowski regression equations; (4) the rational method; and (5) TR-55. A brief description of each of these methods is included in Appendix B.

With the exception of TR-55, these models can only be used to estimate flood peaks and cannot be used to produce hydrographs. TR-55 can be used to produce hydrographs, but is not as detailed a model as EPA-SWMM. TR-55 relies on user-input times of concentration and travel times, whereas EPA-SWMM uses the kinematic wave method and full dynamic equations for overland and conduit routing, respectively. The

five models listed above were used only as general references to gauge against the EPA-SWMM results in the absence of flow data.

The results from the model comparison are shown in figure 2.9. The initial EPA-SWMM model, denoted as "SWMM" in the figure, matches the other models for the 2- and 10-year storms, but has the lowest peak of all models for the 100-year storm. Analysis of the output from the EXTRAN module revealed that, for the higher return period storms, there was significant flooding throughout the watershed. For EPA-SWMM, "flooding" refers to water that has reached the ground surface at a junction and "surcharge" refers to water that has risen above the crest of a pipe in a junction. Any water that is lost through flooding of inlets or channels does not re-enter the conduit network in EPA-SWMM, in effect reducing the peak discharge that enters the pond.

The only way to counter the effects of flooding and subsequent loss of water is to increase the capacity of the culverts and channels modeled either by making them larger, deeper, or by decreasing their roughness factor. Only conduits that were flooded for more than 10 minutes were enlarged and deepened, and they were modified to two standards. One modified model consisted of conduits re-sized such that they would contain the 25-year event, and a second consisted of conduits re-sized for the 100-year storm. These two additional models are shown as "SWMM-25" and "SWMM-100" respectively in figure 2.9. Note that both of these models resulted in flood peaks well above those of most of the other models.

The increased flood peaks for the SWMM-25 and SWMM-100 models are due to the greater volume of water retained in the system. In addition, both the SWMM-25 and SWMM-100 models appear to have resulted in abnormally high flood peaks for the 2- and 10-year storms, likely due to reduced travel times within the artificially enlarged conduits. It is unclear, however, why the 2-year flood peak is greater in the SWMM-25 model than in the SWMM-100 model. The SWMM-100 model contains conduits that are equal or greater in size than those in the SWMM-25 model, so its 2-year flood peak should have been at least equal to, if not greater than, the SWMM-25 peak. It is possible that the increased peak for the 2-year storm in the SWMM-25 model is due to greater surcharge head, and thus increased velocity, at critical junctions in the model.

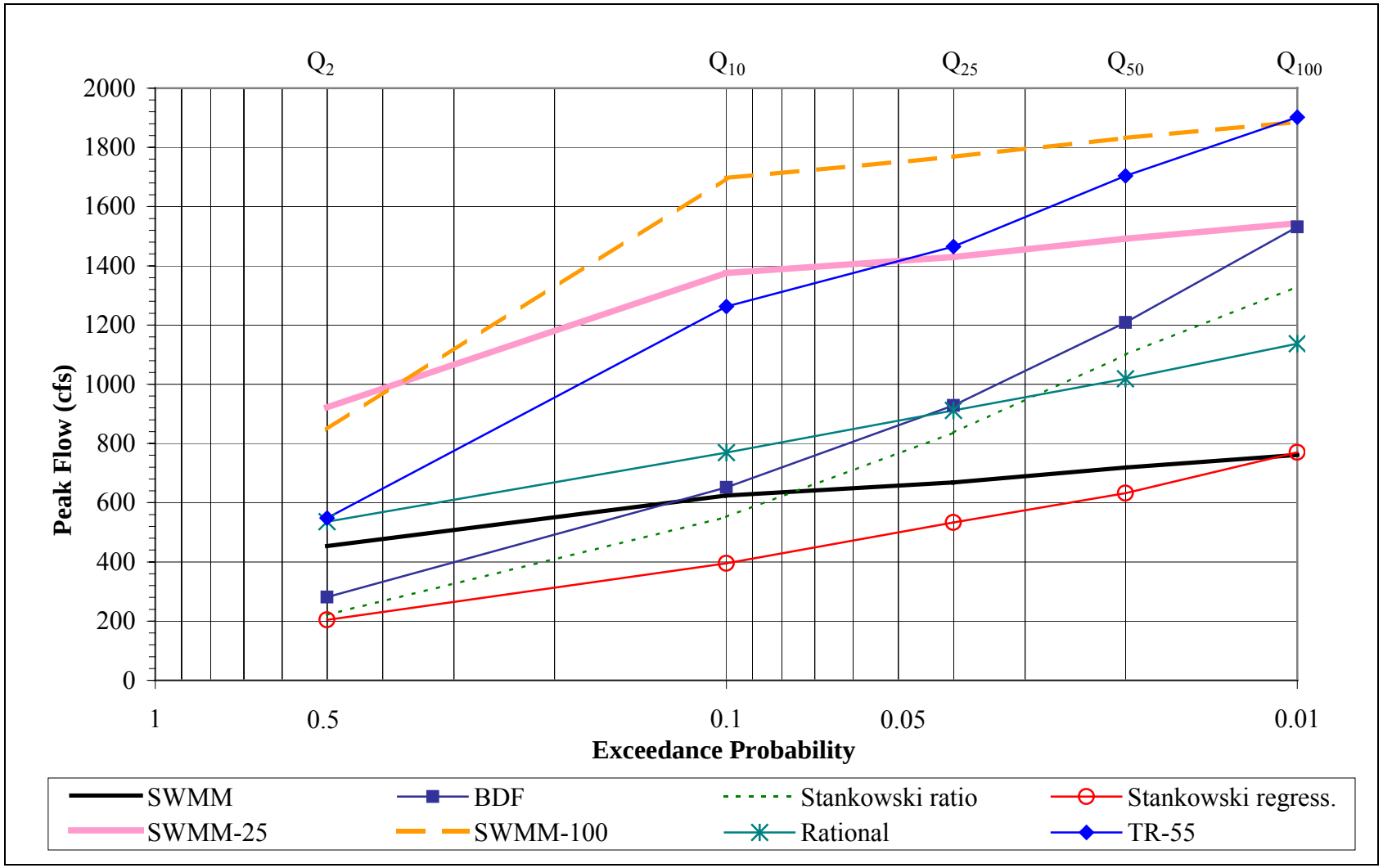

Figure 2.9 – Comparison of flood peaks at the upper pond for eight stormwater models.

Based on the results above, the SWMM-25 model was chosen for the final stormwater model. The three SWMM models were certainly the most detailed of all the models examined and of those three, the SWMM-25 model met or exceeded most of the other flood peaks with the least modification to the conduit network. Since even the 2-year storm caused flooding in the unmodified SWMM model, it is apparent there are several undersized conduits within the watershed. Were these major conduits sized correctly, they would be designed to contain the 25-year storm. Therefore, a conduit network modified to function at that level is a reasonable simplifying assumption.

A final check made on the SWMM-25 model was a comparison against the Hydrologic Modeling System (HEC-HMS), a model created and maintained by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers. This comparison was done in order to check the application of the Green-Ampt infiltration model between the two stormwater models. HEC-HMS was used because it not only allows the user to select the Green-Ampt model for infiltration, but it is also able to use the kinematic wave method for overland flow routing as does SWMM. Table 2.3 lists the runoff depths from each subwatershed for the 2-year storm. Runoff depths were nearly equal for every subwatershed, with a maximum difference of 5.1% in subwatershed #4.

Table 2.3 – Comparison of 2-year runoff depths from HEC-HMS and SWMM.

	Runoff Depth (in)		
Subarea	**HMS**	**SWMM**	**Difference**
1	1.70	1.72	-1.4%
2	1.32	1.35	-2.2%
3	1.15	1.19	-3.7%
4	1.13	1.19	-5.1%
5	1.37	1.41	-2.7%
6	1.47	1.52	-3.5%
7	1.70	1.73	-1.7%
8	1.24	1.29	-4.1%
9	1.32	1.33	-0.9%
10	1.75	1.77	-1.3%
11	1.27	1.28	-0.9%
12	2.26	2.26	0.2%

2.5 ROUTING THROUGH UPPER POND

Although SWMM has the capability to route storms through storage structures, the Virginia Tech version of the Penn State Urban Hydrology Model (VTPSUHM) was used instead for this portion of the project. The Runoff and EXTRAN blocks of SWMM do not allow the use of custom elevation-storage rating curves. The Storage/Treatment block of SWMM is specifically designed for routing flows through storage structures, but the learning curve would have been steep. VTPSUHM does storage routing readily with the modified Puls method (also used by the Storage/Treatment block), and the program was already familiar.

The data inputs required for storage routing are inflow hydrographs, elevation-storage data, and elevation-discharge data. The inflow hydrographs were created using SWMM and then converted to a format usable by VTPSUHM. Elevation-storage data for the upper pond were taken from 1' contour mapping available from the University. This mapping had been developed from aerial photographs taken in 1999, and contained detailed topographic information of the area surrounding the pond. In order to make the model as accurate as possible, additional spot elevations were taken along the top of the dam between the ponds and on both concrete staircase structures. These elevations were acquired with an automatic level and Philadelphia rod in a level loop. The loop was started on a survey monument at the western end of the drillfield[1]. The loop was closed on the same monument to within 0.04' of the original elevation. Elevations were not corrected for this small error.

The spot elevations taken along the dam and staircase structure were used to create the elevation-discharge rating curve for the pond as well as the elevation-storage curve. The staircase structure between the ponds was modeled in VTPSUHM as a series of rectangular weirs. Figure 2.10a shows a photograph of the large staircase structure, and figure 2.10b shows a cross-section of the structure and dam with elevations. Output from VTPSUHM is included in Appendix C.

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[1] Monument GPS-99-11 was set by University surveyors and used for the 1999 aerial topography of the campus. The monument is a bronze disk set in concrete, and is at elevation 2027.54' (NGS NAD 88).

Figure 2.10a – Large staircase structure between the upper and lower ponds.

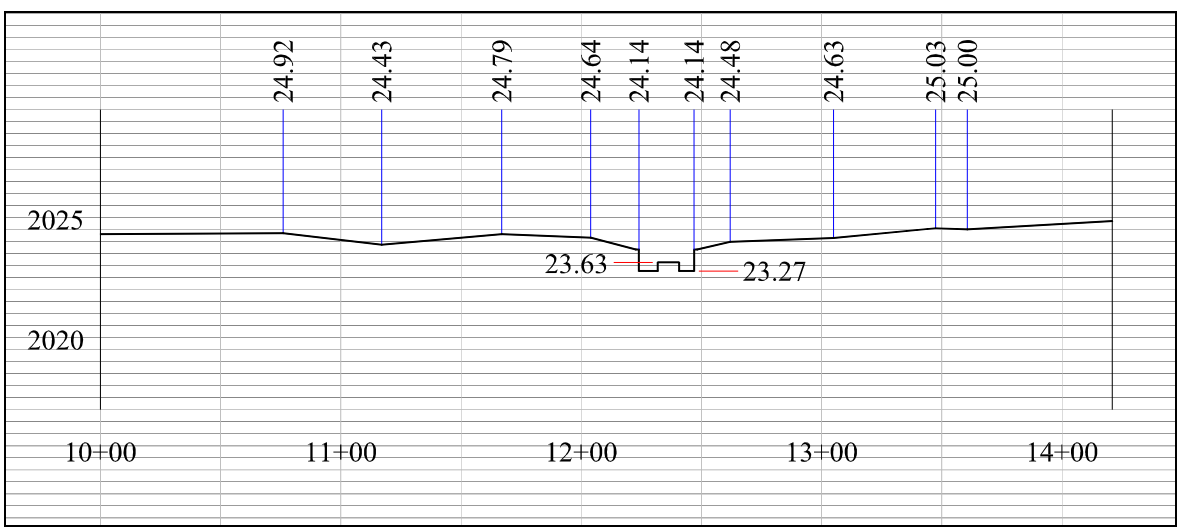

Figure 2.10b – Cross-section of upper pond dam.

2.6 LOWER POND MODEL

In order to determine if backwater from the lower pond would affect the capacity of the low-flow channel, a stormwater model of the lower pond was developed. The drainage basin of the lower pond is significantly larger than that of the upper pond (1,890 acres vs. 715 acres) and is almost entirely developed. The basin encompasses most of Blacksburg and the Virginia Tech campus, and is primarily residential and commercial. Since an EPA-SWMM model of the lower pond basin would be very large and complex to create, a simpler method was chosen: the SCS unit hydrograph.

A unit hydrograph is simply the response from a watershed given a uniform rainfall excess of 1" depth. The shape of the unit hydrograph is dependent on watershed characteristics, particularly surface cover and time of concentration. Hydrographs for any rainfall amount can be produced from the unit hydrograph by multiplying the unit hydrograph ordinates by the desired rainfall depth.

The SCS unit hydrograph is given in table 2.4 and figure 2.11. This unit hydrograph is then scaled to a particular watershed using the watershed's peak flow, Q_p, and time-to-peak, T_p. These variables are calculated using the following equations:

$$T_p = \frac{\Delta D}{2} + T_l \text{ (hours)} \qquad [2\text{-}6]$$

$$Q_p = \frac{484A}{T_p} \text{ (cfs)} \qquad [2\text{-}7]$$

where ΔD is the duration of rainfall excess (min.), T_l is the watershed lag time (hours), and A is the watershed area in square miles. The watershed lag time is defined as the time from the midpoint of rainfall excess to the peak flow, and can be calculated two ways:

$$T_l = 0.6T_c \qquad [2\text{-}8]$$

$$T_l = \frac{L^{0.8}(1000 - 9CN)^{0.7}}{1900CN^{0.7}Y^{0.5}} \qquad [2\text{-}9]$$

where T_c is time of concentration (hours), L is the main channel length (feet), CN is the SCS curve number of the watershed, and Y is the average slope in the watershed (%).

Table 2.4 – SCS dimensionless unit hydrograph ordinates.

T/T_p	Q/Q_p	T/T_p	Q/Q_p	T/T_p	Q/Q_p	T/T_p	Q/Q_p
0.0	0.00	1.4	0.78	2.8	0.077	4.2	0.0100
0.2	0.10	1.6	0.56	3.0	0.055	4.4	0.0070
0.4	0.31	1.8	0.39	3.2	0.040	4.6	0.0030
0.6	0.66	2.0	0.28	3.4	0.029	4.8	0.0015
0.8	0.93	2.2	0.207	3.6	0.021	5.0	0.0000
1.0	1.00	2.4	0.147	3.8	0.015		
1.2	0.93	2.6	0.107	4.0	0.011		

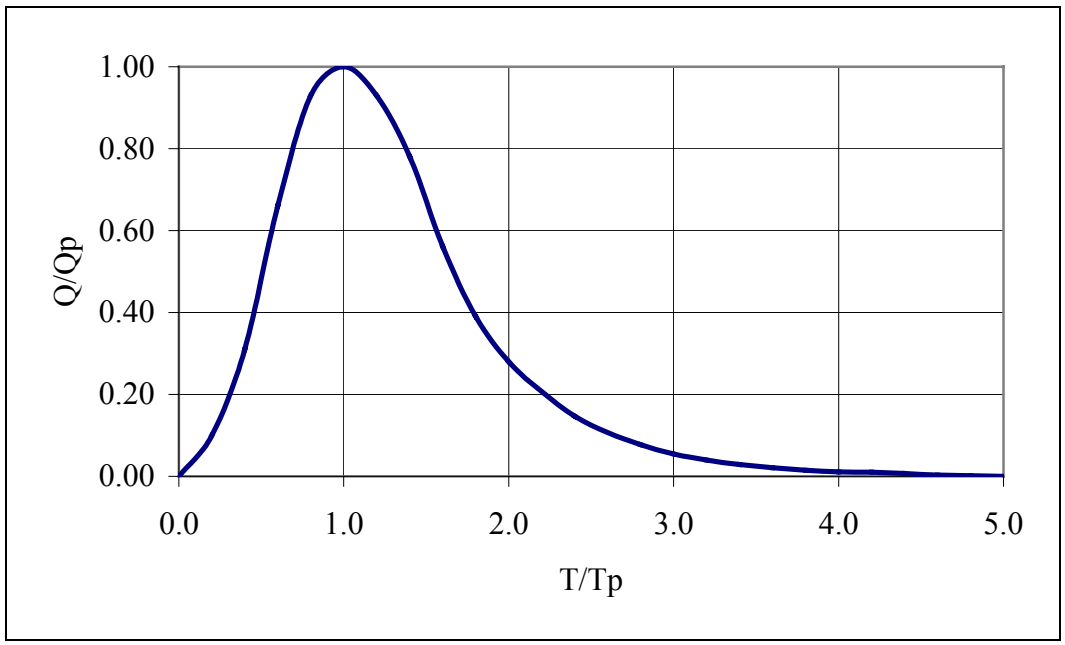

Figure 2.11 – SCS dimensionless unit hydrograph.

A curve number was calculated for the lower pond watershed assuming its impervious fraction would be approximately equal to that of the upper pond watershed (31.9%). A curve number of 98 was used for impervious areas, and a curve number of 74 was used for pervious areas, assuming grass in good condition (Bedient and Huber 1992). These curve numbers were then weighted by area:

$$\overline{CN} = \frac{A_{perv}CN_{perv} + A_{imperv}CN_{imperv}}{A_{total}} = 0.681(74) + 0.319(98) = 81.66 \qquad [2\text{-}10]$$

Since curve numbers must be whole numbers, a curve number of 82 was used for the lower pond watershed.

The length of the main channel was determined using topographic mapping of the Town of Blacksburg. The southern branch of Stroubles Creek begins near Floyd Street, flowing a total of 11,930 feet along the main channel to the lower pond. The average basin slope was calculated in Land Development Desktop using the same method described in section 2.2. The calculated slope was 8.3% using only 10' contours. Therefore, the basin lag time was calculated as:

$$T_l = \frac{L^{0.8}(1000 - 9CN)^{0.7}}{1900CN^{0.7}Y^{0.5}} = \frac{(11{,}930)^{0.8}(1000 - 9\times 82)^{0.7}}{1900(82)^{0.7}(8.3)^{0.5}} = 0.75\ hours \qquad [2\text{-}11]$$

Instead of calculating the hydrographs manually using table 2.4, VTPSUHM was used to produce hydrographs with the SCS unit hydrograph method. The required inputs in VTPSUHM are basin area (mi.^{2}), time of concentration (hours), curve number, unit hydrograph K-factor (484 in equation [2-7]), and an input hyetograph. The basin area was calculated as:

$$A = \frac{1{,}890\ ac}{640\ ac/mi.^2} = 2.95\ mi.^2 \qquad [2\text{-}12]$$

Since VTPSUHM requires a time of concentration instead of a lag time, the time of concentration for the basin was back-calculated from the lag time in equation [2-11]:

$$T_c = \frac{T_l}{0.6} = \frac{0.75\ hours}{0.6} = 1.25\ hours \qquad [2\text{-}13]$$

Hyetograph data were taken directly from the XP-SWMM input files. The hyetographs are for 24-hour storms with data points every 30 minutes. A summary of peak flows for the lower pond is shown in table 2.5. Full hydrograph outputs from VTPSUHM are included in Appendix C.

Table 2.5 – SCS unit hydrograph peak flows and maximum water surface elevations (WSEL) at the lower pond.

Return Period (years)	Peak Flow (cfs)	Maximum WSEL (feet)
2	1102	2022.0
10	2450	2022.9
25	2818	2023.1
50	3263	2023.3
100	3639	2023.5

A rating curve for the concrete overflow spillway at the outlet of the lower pond was developed to estimate the maximum WSEL in the pond for each storm modeled. The rating curve is shown in figure 2.12. Output data from VTPSUHM used to create the curve is included in Appendix C. The maximum WSEL (water surface elevation) for each storm was calculated assuming the peak pond outflow was equal to the peak inflow, e.g. no peak attenuation from storage effects. This was done in order to make the WSEL estimates more conservative. The calculated maximum WSEL for each storm is shown in table 2.5. The water depths for the 2-, 10-, and 25-year storms should not present a backwater problem in the low-flow channel since the lower pond maximum WSEL will be well below the channel invert. Backwater effects may reduce the channel's capacity during the 50- and 100-year events, but any loss of capacity in the channel would cause only a minor increase in the depth of flow over the dam between the ponds.

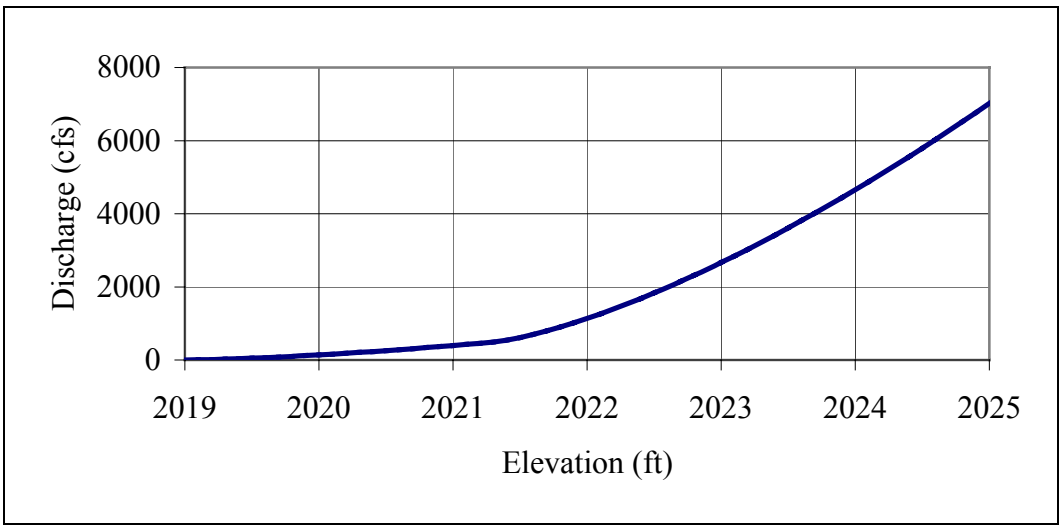

Figure 2.12 – Elevation-discharge rating curve for lower pond spillway.

CHAPTER 3
IMPROVEMENTS TO VTPSUHM FOR USE IN DESIGN

The original design concept for the pond modifications included a riser pipe and outfall culvert between the upper and lower ponds. The design of these structures in VTPSUHM would require routing storms through a riser pipe that would likely fill completely. However, the current version of VTPSUHM does not allow the design of a riser that completely floods, e.g. the water surface elevation (WSEL) inside the riser structure is equal to the WSEL in the basin above it. This limitation was purposely designed into the program to prevent the design of undersized outfall culverts. Still, there are instances where the size of the outfall culvert is limited to that of an existing pipe. Alternatively, in the case of the upper pond, a flooded riser may be unavoidable due to the effects of backwater or minimal available head.

As it turned out, the final design of the outlet between the upper and lower ponds did not include a riser structure. Thus, the flooded riser problem in VTPSUHM became a moot point for the project. Yet before it became apparent that a riser was not feasible for the upper pond, several weeks were spent modifying the VTPSUHM code to remove the full riser limitation and making several other important changes.

For the sake of clarity, only sections of the code that were modified are shown along with several lines of code above and below the changed section for location purposes. Generally, code that was added or changed is shown in bold italics.

3.1 FULL RISER FATAL ERROR

The first step in modifying VTPSUHM to allow a flooded riser was to eliminate the automatic shutdown of the routing program when the WSEL in the riser matched the WSEL in the basin. This shutdown was coded in the form FRMORATE as shown below:

```
3500 If BOXEL(K) <= Stage(I, 1) Then GoTo 3530 Else MsgBox
     "WARNING! OUTLET CULVERT CAPACITY INSUFFICIENT!
     PROGRAM TERMINATED! *****", 0
3510 MsgBox "YOU MUST MODIFY OUTLET STRUCTURE", 0: Exit Sub
```

The array `BOXEL(K)` contains the WSEL within the riser box at step K, and `Stage(I,1)` is the invert elevation of the current outlet structure component. Since these two lines of code are contained within the grate and riser pipe drop inlet flow calculation subroutine, this check is only performed on the uppermost component of the riser structure. To remove the shutdown, these two lines of code were simply "commented out" to make them inactive. When the remaining code was checked for references to these two lines, a reference to line 3510 was found in line 3580 of the emergency spillway flow calculation subroutine as shown below. This code ensures that the tailwater does not flood the emergency overflow channel:

```
3580 If TWEL(K) > Stage(I, 1) Then Print "***** WARNING!
     OUTFALL CHANNEL CAPACITY INSUFFICIENT!  PROGRAM
     TERMINATED! *****": GoTo 3510
```

Since line 3510 had been commented out, part of the code from line 3510 was added to the end of line 3580 to keep the warning message:

```
3580 If TWEL(K) > Stage(I, 1) Then Print "***** WARNING!
     OUTFALL CHANNEL CAPACITY INSUFFICIENT!  PROGRAM
     TERMINATED! *****": MsgBox "YOU MUST MODIFY OUTLET
     STRUCTURE", 0: Exit Sub
```

3.2 OUTFALL CULVERT CONTROL OF RISER OUTFLOW

When the riser structure is completely flooded from the inside, all outflow from the pond will be controlled by the outfall culvert. The orifices and other openings in the riser cannot be controlling the outflow under this condition because the WSEL inside the riser equals the WSEL in the pond and all openings are submerged from the inside. VTPSUHM calculates the elevation-outflow rating curve for the entire outlet structure at a regular interval between the minimum and maximum pond elevation (input by the user). To check whether the outfall culvert is controlling the pond outflow, code was added to calculate the outfall culvert capacity at each interval, using the pond WSEL to calculate head on the culvert. In reality, the WSEL inside the riser would be less than that in the pond up until the point when the outfall culvert begins to control the pond outflow. From that point, the WSEL inside the riser would be equal to that in the pond. However, since the outfall culvert would not be controlling pond outflow if the riser box

WSEL were less than the pond WSEL, the culvert capacity as calculated above must be less than the riser structure capacity. Essentially, what is calculated in the new code is the outfall culvert capacity, *assuming* that the riser box is completely flooded.

A preferable method of calculating the culvert capacity would use the riser box WSEL instead of the pond WSEL. However, the nature of calculating the riser box WSEL is inherently unstable, and calculating the culvert capacity using the riser box WSEL seemed only to compound this instability. In order to calculate the WSEL inside the riser box at each pond WSEL interval, VTPSUHM must use an iterative calculation to converge on a result. This iteration is necessary because the riser box WSEL and the riser box outflow capacity are dependent on one another. Therefore, the basin WSEL was used to calculate the potential culvert capacity instead of the actual culvert capacity.

The following code was added to FRMORATE to calculate the potential culvert capacity at a riser box WSEL equal to that in the basin. This code uses the same method to calculate the culvert capacity as the subroutine that calculates the capacity of a discharge pipe:

```
5100 Rem ********* OUTFALL CULVERT CAPACITY CALCULATION
     ***********
5110 If BOXEL(K) > Stage(0, 1) Then OUTFH(K) = BOXEL(K) -
     Stage(0, 1) Else OUTFQ(K) = 0: GoTo 5220
5120 OUTFHWD(K) = OUTFH(K) / Stage(0, 2)
5130 If OUTFHWD(K) <= 0.6 Then OUTFQ(K) = Stage(0, 2) ^ 2 *
     Sqr(32.2 * Stage(0, 2)) * 0.48 * (OUTFHWD(K)) ^ 1.9:
     GoTo 5200
5140 If OUTFHWD(K) > 5 Then OUTFQ(K) = OUTFQ(K - 1) *
     Sqr(OUTFH(K) / (OUTFH(K) - ELINT)): GoTo 5200
5150 If Entrance(0) = "SOH" Then GoTo 5170
5160 OUTFQ(K) = Stage(0, 2) ^ 2.5 * (4.701 * OUTFHWD(K) -
     1.09 * OUTFHWD(K) ^ 2 + 0.1248 * OUTFHWD(K) ^ 3 -
     0.0007 * OUTFHWD(K) ^ 5 - 1.4601): GoTo 5200
5170 OUTFQ(K) = Stage(0, 2) ^ 2.5 * (5.433 * OUTFHWD(K) -
     1.286 * OUTFHWD(K) ^ 2 + 0.1608 * OUTFHWD(K) ^ 3 -
     0.00125 * OUTFHWD(K) ^ 5 - 1.8007)
```

```
5200 OUTFALTQ = ((BOXEL(K) - CULVTWEL) * 39.7252 * Stage(0,
     2) ^ 4 / (184.139 * Stage(0, 8) ^ 2 * Stage(0, 6) /
     Stage(0, 2) ^ 1.333 + Ent2 + EX)) ^ 0.5
5210 If OUTFALTQ < OUTFQ(K) Then OUTFQ(K) = OUTFALTQ
5220 Return
```

Code was also added to FRMORATE to define the new variables used in the code above:

```
Static CROWNEL(150) As Single
Static Control As String
Static OUTFH(150) As Single
Static OUTFHWD(150) As Single
Static OUTFQ(150) As Single
Static SPQSUM(150) As Single
Static CULVOVRD(150) As Integer
Static OVRDTXT As String
```

Descriptions of the variables were also added to FRMORATE:

```
2440 Rem   PERIM(0)       "      "   wetted perimeter, ft.
2450 Rem   NME$      Outlet stage/discharge rating table
     file name
2460 Rem   CD(0)     Iteration trap counter
2461 Rem   OUTFQ(K)  Outfall culvert potential outflow at
     WSEL j, cfs
2462 Rem   OUTFH     Potential head on outfall culvert, ft.
2463 Rem   OUTFHWD   Outfall culvert head / diameter ratio,
     ft/ft.
```

Line 2605 sets SPQSUM to the outflow of all emergency spillways at elevation K:

```
2600 QCALC(K) = QCALC(K) + Q(I, K) + SPQ(I, K)
2605 SPQSUM(K) = SPQSUM(K) + SPQ(I, K)
2610 If I < NSTAGE Then GoTo 2530
2620 If Outfall = 1 Then GoTo 2670
```

Lines 2684 through 2688 compare OUTFQ to the principal spillway capacity. If the culvert capacity is less than the spillway capacity, the culvert capacity is used instead. The variable CULVOVRD is used to display text in the output alerting the user when the culvert is controlling outflow from the riser structure:

```
2673 GoTo 2680
2675 If DPC = 0 Then SPQ(0, K) = 0 Else SPQ(0, K) = DPC
2680 If QCALC(K) = 0 Then SPQ(0, K) = 0
2684 QSUM(K) = QCALC(K)
2685 If Outfall < 3 Then CULVOVRD(K) = 0: GoTo 2690
2686 GoSub 5100
2687 CULVOVRD(K) = 1
2688 If QSUM(K) - SPQSUM(K) - OUTFQ(K) > 0.1 Then QSUM(K) =
     SPQSUM(K) + OUTFQ(K): CULVOVRD(K) = 2
2690 Next j: Rem PRINT RESULTS
```

A fifth column was added to the output screen for the CULVOVRD text:

```
Grid1.ColWidth(4) = 1600
Grid1.ColWidth(5) = 1200
Grid1.RowHeight(0) = 600
```

Header text for the fifth column:

```
Grid1.Col = 4
Grid1.Text = "Outfall" + Chr$(13) + "Culvert" + Chr$(13) +
     "Control Type"
Grid1.Col = 5
Grid1.Text = "Outfall" + Chr$(13) + "Culvert" + Chr$(13) +
     "Override?"
For j = ELMIN To ELMAX + 0.01 Step ELINT
Grid1.Row = K
Grid1.Col = 0
```

Finally, code was added to the print subroutine to add the CULVOVRD column. In addition, the outfall culvert flow control text was changed from “ORIFICE” and “FRICTION” to “INLET” and “OUTLET”, respectively:

```
If SPQ(0, K) = 0 Then
    Control = "N/A"
ElseIf SPQ(0, K) = 1 Then
    Control = "INLET"
Else
    Control = "OUTLET"
End If
If CULVOVRD(K) = 0 Then
    OVRDTXT = "N/A"
ElseIf CULVOVRD(K) = 1 Then
    OVRDTXT = "NO"
Else
    OVRDTXT = "YES"
End If
```

Figures 3.1a and 3.1b show the outlet structure configuration and rating curve for a test structure from the previous version of VTPSUHM (version 5.0). Figure 3.2 shows the output for the same structure from the revised VTPSUHM (beta version 6.0). Note that the configuration and rating curve have been fit onto the same page in the new version of VTPSUHM.

Outlet Structure Configuration

Outlet Structure Configuration for Test structure

Stage 1: Circular Orifice

Invert Elevation = 100 feet
Diameter = 1.5 feet
Discharge Coefficient = .6

Stage 2: Rectangular Weir

Crest Elevation = 104 feet
Length = 4 feet
Discharge Coefficient = 3.1

Stage 3: Outfall Culvert

Invert Elevation = 99.9 feet
Pipe Diameter = 2.5 feet
Pipe Length = 50 feet
Pipe Slope = .02 ft/ft
Manning n = .013
Entrance Condition = SEH
Number of Barrels = 1

Figure 3.1a – Outlet structure configuration for test structure (VTPSUHM version 5.0).

Basin Rating Curve

Outlet Rating Table for Test structure

Basin Water Elevation	Basin Outflow (cfs)	Riser Box Water Elevation	Tailwater Elevation (ft)	Outfall Culvert Control
100.00	0.00	99.90	N/A	N/A
100.50	0.80	100.28	N/A	ORIFICE CONT
101.00	2.78	100.65	N/A	ORIFICE CONT
101.50	5.49	100.98	N/A	ORIFICE CONT
102.00	6.64	101.09	N/A	ORIFICE CONT
102.50	8.75	101.28	N/A	ORIFICE CONT
103.00	10.49	101.42	N/A	ORIFICE CONT
103.50	11.95	101.53	N/A	ORIFICE CONT
104.00	13.15	101.61	N/A	ORIFICE CONT
104.50	18.10	101.92	N/A	ORIFICE CONT
105.00	26.30	102.36	N/A	ORIFICE CONT
105.50	36.77	102.84	N/A	ORIFICE CONT
106.00	47.71	103.85	N/A	ORIFICE CONT
106.50	55.86	104.87	N/A	ORIFICE CONT
107.00	72.94	104.87	N/A	FRICTION CONT
107.50	80.16	105.75	N/A	FRICTION CONT
108.00	87.37	106.62	N/A	FRICTION CONT
108.50	94.59	107.49	N/A	FRICTION CONT
109.00	101.81	108.37	N/A	FRICTION CONT
109.50	109.02	109.24	N/A	FRICTION CONT
110.00	116.24	110.12	N/A	FRICTION CONT

Figure 3.1b – Basin rating curve for test structure (VTPSUHM version 5.0).

Outlet Structure Configuration

Outlet Structure Configuration for Test structure

Stage 1: Circular Orifice
Invert Elevation = 100 feet
Diameter = 1.5 feet
Discharge Coefficient = 0.6

Stage 2: Rectangular Weir
Crest Elevation = 104 feet
Length = 4 feet
Discharge Coefficient = 3.1

Stage 3: Outfall Culvert
Invert Elevation = 99.9 feet
Pipe Diameter = 2.5 feet
Pipe Length = 50 feet
Pipe Slope = 0.02 ft/ft
Manning n = 0.013
Entrance Condition = SEH
Number of Barrels = 1

Basin Rating Curve

Basin Water Elevation	Basin Outflow (cfs)	Riser Box Water Elevation	Tailwater Elevation (ft)	Outfall Culvert Control	Outfall Culvert Override?
100.00	0.00	99.90	N/A	N/A	NO
100.50	0.80	100.28	N/A	INLET	NO
101.00	2.78	100.65	N/A	INLET	NO
101.50	5.49	100.98	N/A	INLET	NO
102.00	6.64	101.09	N/A	INLET	NO
102.50	8.75	101.28	N/A	INLET	NO
103.00	10.13	101.42	N/A	INLET	YES
103.50	11.59	101.53	N/A	INLET	YES
104.00	12.72	101.61	N/A	INLET	YES
104.50	16.72	101.92	N/A	INLET	YES
105.00	22.04	102.36	N/A	INLET	YES
105.50	27.29	102.84	N/A	INLET	YES
106.00	36.83	103.85	N/A	INLET	YES
106.50	44.86	104.87	N/A	INLET	YES
107.00	44.86	104.87	N/A	OUTLET	YES
107.50	50.60	105.75	N/A	OUTLET	YES
108.00	55.61	106.62	N/A	OUTLET	YES
108.50	60.08	107.49	N/A	OUTLET	YES
109.00	64.19	108.37	N/A	OUTLET	YES
109.50	68.07	109.24	N/A	OUTLET	YES
110.00	71.81	110.12	N/A	OUTLET	YES

NOTE: When a 'YES' appears in the Outfall Culvert Override column, the outfall culvert is restricting the pond outflow. The Basin Outflow at these data points is equal to the outfall culvert capacity at that riser box water elevation.

Figure 3.2 – Outlet structure configuration and basin rating curve for test structure (VTPSUHM beta version 6.0).

3.3 RATING CURVE INSTABILITY

Several attempts were made at reducing or eliminating the instability problem that sometimes occurs in the generation of elevation-discharge rating curves when the riser is fully or partially submerged because of an undersized outfall culvert. At each basin WSEL step, VTPSUHM calculates a riser box WSEL from the riser inflow. Using the calculated riser box WSEL, the program then recalculates the riser inflow capacity. Then the program calculates the riser box WSEL from the new riser inflow capacity. This process repeats until the results converge (to within 0.1 cfs or 1% difference). An instability in the calculations sometimes occurs when an orifice or discharge pipe in the riser structure becomes completely flooded and the flow regime through the opening changes from weir to orifice flow. At this changeover point, the riser box WSEL may not converge. When this instability appears, the program will double the elevation interval and recalculate all points on the rating curve in an attempt to circumvent the instability. Sometimes the program may double the interval several times, and the instability may sometimes prevent the program from calculating any elevation-discharge data whatsoever.

Two general approaches were attempted to reduce the frequency of instability problems. The first approach involved altering the parameters by which the program determined whether instability was present. The parameters that were examined were the number of calculation iterations and the number of averages the program would perform before changing the elevation interval. Neither of these parameters appeared to have any effect on the frequency of instability problems. After consulting with Scott Chamberlain, one of the original programmers of VTPSUHM, it was determined that the instability parameters were adequate and did not require further alteration.

The second approach provided the program with a smaller elevation interval to use for calculation purposes (0.1*ELINT), and then only the data points at the original elevation interval would be displayed. Initial testing revealed that using a very small elevation interval around the changeover point appeared to eliminate any instability problems. However, when this change was coded into the program, instability became even more frequent, often to the point of program shutdown. It is unclear why this

approach did not work. Further testing is needed since this approach, in theory, should be successful in eliminating the instability problem.

3.4 RETURN TO MSRM MAIN MENU

A minor flaw in the previous version of VTPSUHM appeared on exiting either the routing routine or the basin volume estimating routine in the MSRM module. Instead of being taken back to the MSRM main menu, all windows were closed. To fix this problem, two lines of code were added to FRMPULS:

```
Private Sub Command2_Click()
    Unload FRMPULS
    MSRM1FRM.Show
End Sub

Private Sub Form_Activate()
If UnloadPuls = -1 Then
    Unload FRMPULS
    MSRM1FRM.Show
End If
```

One line of code was also added to FRMPULS2:

```
Private Sub Command3_Click()
    Erase TOTQOUT
    Unload FRMPULS
    Unload FRMPULS2
    MSRM1FRM.Show
End Sub
```

3.5 MINIMUM, MAXIMUM, AND INCREMENT ELEVATIONS

In order to simplify the creation of a basin outlet rating curve (e.g. discharge-elevation curve), code was added to FRMORATE so that the minimum, maximum, and increment elevations used to create the current rating curve are saved as part of the outlet structure file. Code was also added so that these elevations automatically appear in the

first dialog box. If the elevations are changed in the dialog box, then the new elevations appear the next time a rating curve is created. The following code was added to the Command3 subroutine of FRMORATE so that the elevations are written to the structure file when saved:

```
Write #1, Outfall
Write #1, Twel1, Chanbotel, Chanbw, Chanlss, ChanRss,
      Chansl, Chann
Write #1, Stage(0, 1), Stage(0, 2), Stage(0, 6), Stage(0,
      7), Stage(0, 8), Entrance(0), NBar
Write #1, ELMIN, ELMAX, ELINT
```

The next section of code was added to FRMOUTST so that these elevations are read into memory when an existing structure file is opened:

```
Input #1, Outfall
Input #1, Twel1, Chanbotel, Chanbw, Chanlss, ChanRss,
      Chansl, Chann
Input #1, Stage(0, 1), Stage(0, 2), Stage(0, 6), Stage(0,
      7), Stage(0, 8), Entrance(0)
If Not EOF(1) Then
    Input #1, NBar
End If
If Not EOF(1) Then
    Input #1, ELMIN, ELMAX, ELINT
End If
```

CHAPTER 4
MODEL APPLICATION AND PRELIMINARY DESIGNS

4.1 DESIGN GOALS AND INITIAL CONCEPTS

As mentioned in Chapter 1, the primary goal of this project is to create a water quality facility consisting of two ponds in series. To meet this goal, all flow in the existing bypass channel along Duck Pond Drive and all flow from the northern branch of Stroubles Creek must be directed into the upper pond. In particular, small, frequent storms must be contained in the upper pond since many pollutants enter surface streams during the "first flush" that occurs soon after rainfall begins. Therefore, the bypass channel must be closed off and the dam must be modified in order to protect it from frequent overtopping and possible washout due to increased inflow. In addition, the volume of the upper pond should be maximized so that residence time will also be maximized. Finally, all design modifications should consider Solitude and protect it from flood damage.

At the entrance to the bypass channel, the northern branch of Stroubles Creek makes a 45° turn to the left into the upper pond. Once all flow is diverted to the upper pond, there must be some structural measure in place to ensure the entire flow will make this sharp turn instead of continuing straight ahead into the former bypass channel. The original concept for this structure called for a curved stone wall, approximately 3' in height, to direct the stream around this corner. The wall would be approximately 30' in length, and would taper down to a height of 1' at either end.

The initial design concept for modifying the dam consisted of an open riser structure in the western corner of the pond, near the existing standpipe. This riser would then be connected to a large pipe through the existing dam into the lower pond. The riser structure would be sized so that the 2- and 10-year storms could pass through it without overtopping the dam. The riser would likely be designed as a multiple stage inlet with rectangular weir openings in order to accommodate both the 2- and 10-year storms. The riser could also be designed with rocks or other landscaping to hide it from public view while still allowing maintenance access. To protect the dam from larger, less frequent

storms, the dam would be armored with a system of interlocking concrete pavers. A thin layer of topsoil would be applied over the blocks, and then grass planted over the entire dam. The covering of topsoil and grass would help to keep the block in place, and would hide the block from view.

Once the inflow hydrographs had been developed for the upper pond and all modeling information had been entered into VTPSUHM, some initial modeling was done to check the feasibility of the design outlined above. The proposed riser structure was modeled as a rectangular weir with its invert at elevation 2023.5' (just above normal pool), and the 2-year storm was routed through the structure. However, due to the minimal head available below the top of dam at elevation 2024.5', a rectangular weir would have to be nearly 250' long to prevent overtopping the dam. That length equates to a rectangular riser box more than 60' on a side, or a circular riser 80' in diameter. Even if the dam were raised a foot to elevation 2025.5', the weir would need to be 55 feet long. However, raising the dam one foot or more would not be feasible. Since the existing trees on the dam are unable to tolerate that much earth fill over their roots, live wells would be required around each tree, and the additional head generated would greatly increase flows over the existing staircase structure, almost certainly causing severe erosion.

4.2 DAM DESIGN OPTIONS

Three alternative concepts were developed once the original design concept for the dam was determined to be infeasible:

1. Armor the entire dam with interlocking block
2. Create a broad, shallow channel for high frequency, low-flow storms on one side of the dam
3. Create a low-flow channel as above, with an additional staircase structure on the opposite side of the dam

These alternative designs were then tested for technical and economic viability, as well as their adequacy in terms of meeting the design goals.

The first alternative design would involve minor grading along the entire length of the dam (approximately 300 l.f.) to even out the top of the dam to one elevation. The

dam would then be armored using interlocking concrete pavers, covered with a thin layer of topsoil and seeded. These pavers would extend down into the subgrade on all sides (figure 4.1) to protect the edges from being undermined and lifted. Filter fabric would be used underneath the pavers to prevent the migration and loss of topsoil through the pavers. The pavers would not be visible once grass has been established over them, and the grass root system would help to keep the pavers locked together.

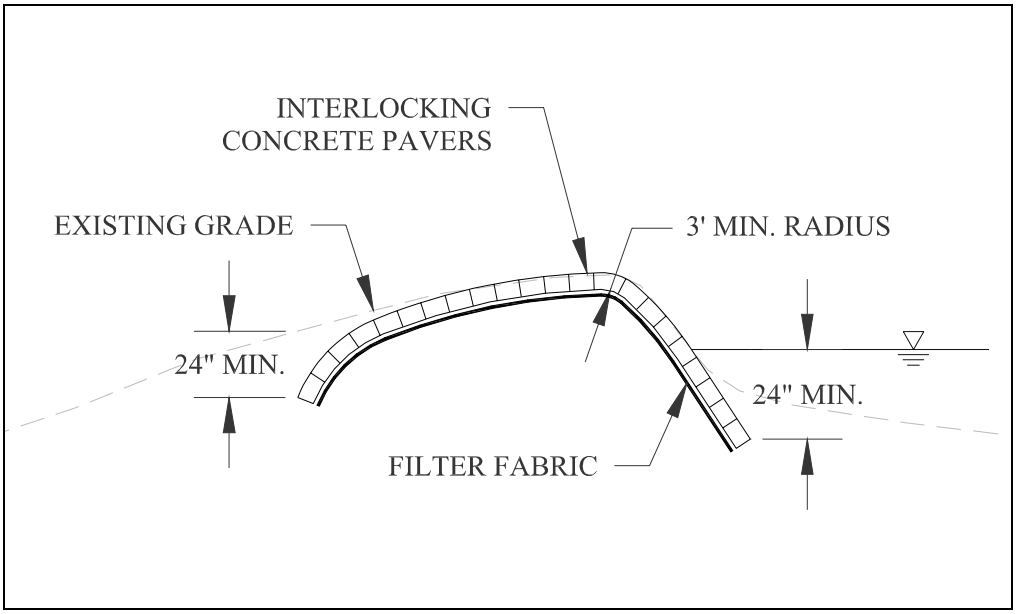

Figure 4.1 – Schematic of interlocking block dam armor.

The second design alternative would involve regrading the dam to one elevation, as in the first alternative. However, the second alternative design would also include a wide section of the dam at a slightly lower elevation, just above the normal pool as shown in figure 4.2. The lowered section would allow water to flow through a broad open channel into the lower pond. This armored channel would be designed to carry high-frequency storms such as the 2-year and smaller storms so that water would flow over the dam less often compared to the first alternative. Since the dam would be overtopped less frequently, it could be armored with a structural material less expensive than the interlocking block.

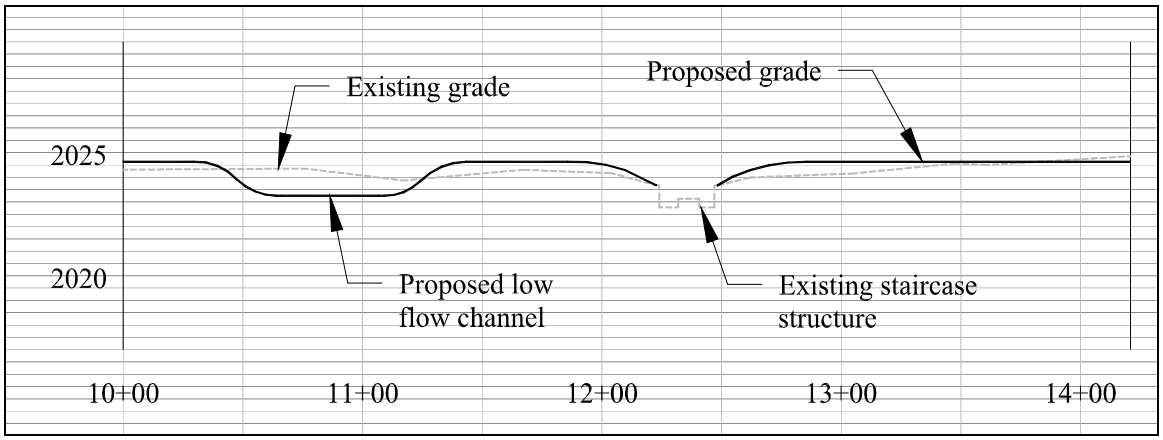

Figure 4.2 – Schematic of design alternative #2 at dam looking downstream.

The third alternative is the same as the one described above, with the addition of a second concrete staircase structure located near the existing standpipe in the southwest corner of the upper pond. The second staircase structure would increase the amount of flow passed through the dam before it would be overtopped. It would also serve the same functions as the existing staircase structure: conveyance of base flow and aeration.

4.3 DESIGN PROCESS AND DECISION-MAKING ON FINAL DAM DESIGN

In addition to the three design alternatives described above, four more alternatives were developed by adjusting component elevations. Descriptions of all seven design alternatives that were tested are listed below. Note that all design alternatives include the existing concrete staircase structure.

<u>Alternative #1</u>

- Entire dam (300’ length) at elevation 2024.50’

<u>Alternative #2</u>

- Low flow channel (100’ wide) at elevation 2023.50’
- Remaining dam (200’ length) at elevation 2025.10’

Alternative #3

- Additional staircase structure (30’ wide) at elevation 2023.30’
- Low flow channel (100’ wide) at elevation 2023.50’
- Remaining dam (170’ length) at elevation 2025.10’

Alternative #4

- Entire dam (300’ length) at elevation 2025.00’

Alternative #5

- Entire dam (300’ length) at elevation 2024.25’

Alternative #6

- Low flow channel (100’ wide) at elevation 2023.50’
- Remaining dam (200’ length) at elevation 2024.50’

Alternative #7

- Additional staircase structure (30’ wide) at elevation 2023.30’
- Low flow channel (100’ wide) at elevation 2023.50’
- Remaining dam (170’ length) at elevation 2024.50’

All seven design alternatives were then routed through the pond using the Modified Puls routing routine in VTPSUHM. The 2-, 10-, and 100-year storm events were used for the preliminary design tests. Table 4.1 lists the maximum water surface elevation reached for each design alternative for the given storms. However, since the dam elevations differ between alternatives, the maximum depth of water over the top of the dam for each alternative is listed in table 4.2. Note that the negative result for the 2-year storm in alternative #3 indicates that the maximum water surface is below the top of the dam.

Table 4.1 – Maximum water surface elevation (ft.) for seven design alternatives.

Return Period	Design Alternative #						
	1	**2**	**3**	**4**	**5**	**6**	**7**
2	2025.24	2025.10	2024.89	2025.63	2025.04	2024.92	2024.80
10	2025.52	2025.45	2025.28	2025.91	2025.32	2025.21	2025.10
100	2025.61	2025.56	2025.39	2026.01	2025.41	2025.30	2025.19

Table 4.2 – Maximum water depth (ft.) over dam for seven design alternatives.

Return Period	Design Alternative #						
	1	**2**	**3**	**4**	**5**	**6**	**7**
2	0.74	0.00	-0.21	0.63	0.79	0.42	0.30
10	1.02	0.35	0.18	0.91	1.07	0.71	0.60
100	1.11	0.46	0.29	1.01	1.16	0.80	0.69

Alternatives #1, #4, and #5 result in the greatest depths of water over the dam, though all three would also entail armoring the entire dam. Alternatives 2 and 3 result in the least depth of water over the dam, but require the greatest fill depth on the dam (approximately 7" maximum). Alternatives 6 and 7 would require no fill over the dam, but result in 4" or greater flow depth over the dam for the 2-year storm.

The area of interlocking block required to armor the entire dam would be approximately 29,000 s.f. If only the top of the dam were armored (approximately 40' width by 300' length), the area of block required would be 12,000 s.f. The area of the low flow channel is approximately 9,800 s.f., one third of the area required for armoring the entire dam.

Once these preliminary findings were completed, the Capital Design and Construction departments of Virginia Tech were consulted in a field meeting at the pond on November 4th, 2002. All design alternatives were described and discussed thoroughly, primarily from the perspective of constructability and cost. Design alternatives 1, 4, and 5 were quickly ruled out due to the high cost of interlocking block. Alternatives 3 and 7 were similarly ruled out due to the cost of installing a second staircase structure. Since

alternative #6 allowed frequent overtopping of an unprotected dam, alternative #2 was left as the remaining option. There was concern that the low-flow channel would not blend in with the surrounding landscape. It was explained that the channel would only be 1.4' deep and 50' wide at the deepest point, and that shallow side slopes (6H:1V) would be used to prevent the appearance of a sharply-defined channel. Additionally, once the blocks had been filled with topsoil and seeded, they would not be visible through the grass. The consensus reached was that alternative #2 should be used for the final design.

4.4 CHANNEL STABILIZATION

Another design aspect of this project is the stabilization of the inflow channels into both the upper and lower ponds. These channels have become severely eroded and several large trees are threatened by undercutting. Riprap and crusher run had been placed along the banks of the northern branch of Stroubles Creek between West Campus Drive and Duck Pond Drive in an attempt to halt erosion, but this effort has succeeded only marginally. Interlocking concrete pavers will be installed in order to stabilize the banks of both streams as part of this project.

Figure 4.3 is a schematic cross-section of a stream bank where the interlocking block will be used. On both sides of the stream and at the streambed, the block will be turned down into the existing grade a minimum of 24" to avoid undercutting and uplift of the block. Filter fabric will be used underneath the block to prevent migration and loss of topsoil, while still allowing percolation of rainfall. Block is only placed along the banks of the stream, and not along the bottom. Placing block along the streambed would disrupt the benthic ecosystem, eliminating habitat for vertebrates and invertebrates, and would be unnecessary and wasteful besides, as the streambed is not eroding as severely as the stream banks.

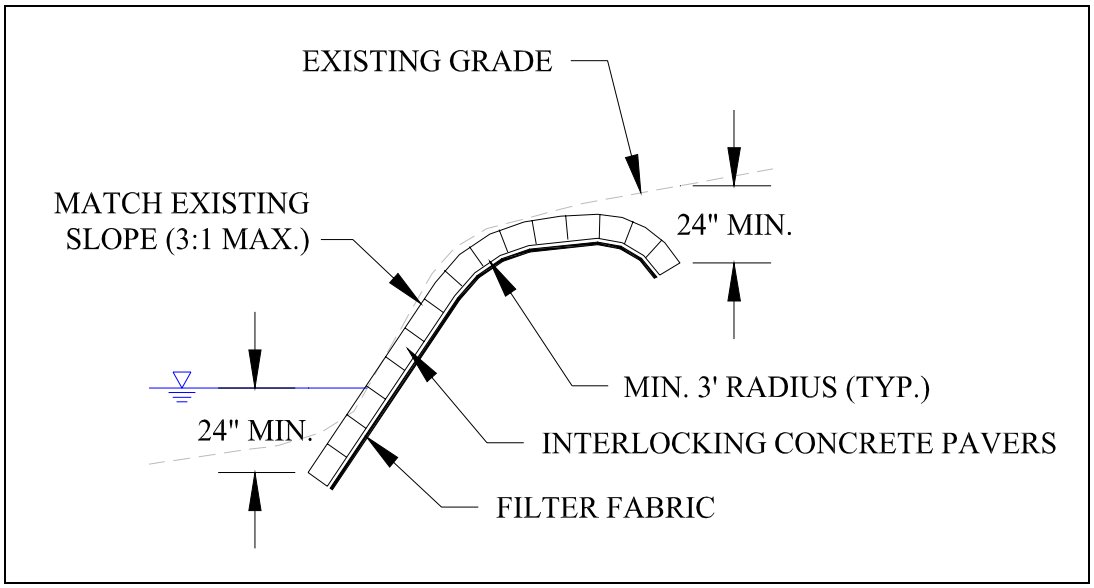

Figure 4.3 – Schematic of stream bank stabilization with interlocking pavers.

4.5 WEIR DESIGN AND CALIBRATION

As part of a future water quality improvement project, flow and water quality monitoring equipment will be installed on both branches of Stroubles Creek as they enter the upper and lower ponds. The design, testing and installation of concrete weirs for the flow monitoring equipment are part of this project. Two weirs will be installed, one at the box culvert crossing West Campus Drive above the upper pond, and the other underneath the pedestrian bridge below the drillfield. Both weirs will be of the same basic design: a rectangular low-flow opening in the center, and a 150° V-notch extending to vertical sides (figure 4.4). The low-flow opening will allow a greater base flow to pass through compared to a simple V-notch weir, and it will prevent sediment from building up behind the weir. The edges of the 150° V-notch will be tapered as shown in figure 4.5. The original design for the weirs called for a 120° angle (dashed line in figure 4.4), but this angle would have constricted the openings severely and would have allowed too little flow to pass through.

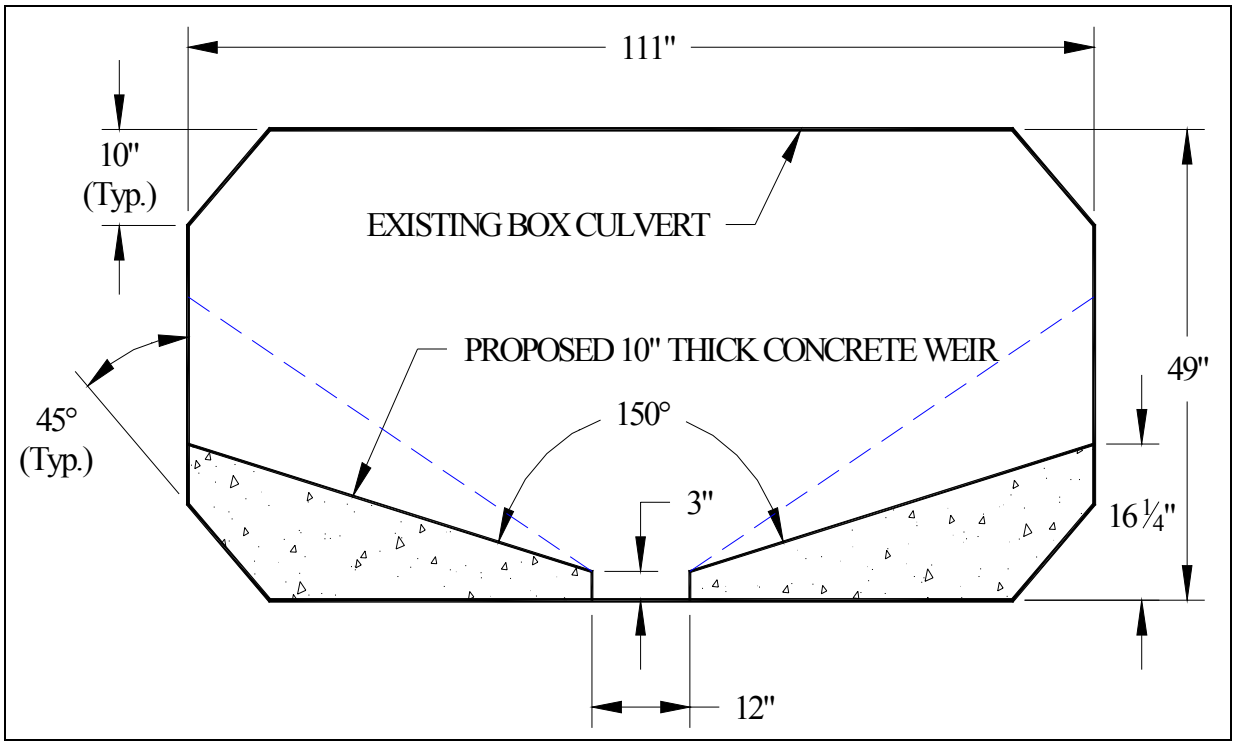

Figure 4.4 – Proposed weir at box culvert.

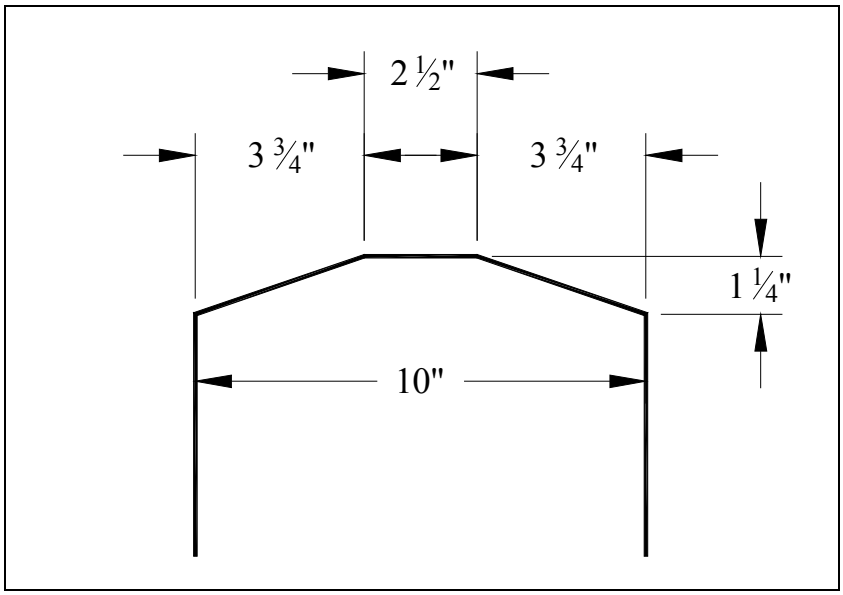

Figure 4.5 – Cross-section of 150° V-notch.

The flow monitoring equipment to be installed at both weir sites will measure depth of flow instead of measuring flow directly. The recorded depths will then be used to calculate flow with the use of a depth-flow relationship. The equations describing the

depth-flow relationship for each weir are calculated using scale models of the weirs. A model is placed in a hydraulic flume where water can be pumped through an open channel at known flow rates. At different flow rates, the depth of water over the scaled weir is measured and a depth-flow relationship is developed for the model. Then using non-dimensional analysis, or similitude, a depth-flow relationship for the full-size (prototype) weir can be extrapolated from the model data.

4.5.1 SIMILITUDE

For open channels, two systems can be said to be hydraulically similar if the Froude numbers (F_r) of the two systems, the model and the prototype, are equal:

$$(F_r)_m = (F_r)_p \qquad [4\text{-}1]$$

Since $F_r = \frac{v}{\sqrt{gL}}$ where v is velocity, g is the gravitational constant and L is length, then:

$$\left(\frac{v}{\sqrt{gL}}\right)_m = \left(\frac{v}{\sqrt{gL}}\right)_p \qquad [4\text{-}2]$$

Then, if $r = \frac{m}{p}$ such that $L_r = \frac{L_m}{L_p}$, etc., then [4-2] can be rewritten:

$$\frac{v_r}{\sqrt{g_r L_r}} = 1 \qquad [4\text{-}3]$$

Assuming that $g_r = 1$ or $g_m = g_p$, then [4-3] can be further simplified:

$$v_r = \sqrt{g_r L_r} = \sqrt{L_r} \qquad [4\text{-}4]$$

Therefore, the ratio of flow is:

$$Q_r = \frac{Q_m}{Q_p} = \frac{v_m A_m}{v_p A_p} = \frac{v_m (L_m)^2}{v_p (L_p)^2} = v_r L_r^2 = L_r^{0.5} L_r^2 = L_r^{2.5} \qquad [4\text{-}5]$$

where Q is flow rate and A is area. For the box culvert, the length ratio, L_r, would be:

$$L_r = \frac{L_m}{L_p} = \frac{12''}{111''} = 0.1081 \qquad [4\text{-}6]$$

and so the flow ratio, Q_r, would be:

$$Q_r = L_r^{2.5} = (0.1081)^{2.5} = 3.843 \times 10^{-3} \qquad [4\text{-}7]$$

The length and flow ratios are then used to transform the depth-flow relationship found in the lab to a depth-flow relationship for the prototype.

4.5.2 MODEL CONSTRUCTION

Since the weir models need to be scale representations of the prototype weirs, the prototype dimensions were simply scaled down using the length ratio as calculated above. Figure 4.6 shows the plan drawing for the weir model #1 (box culvert weir), to be constructed from 1" thick Plexiglas.

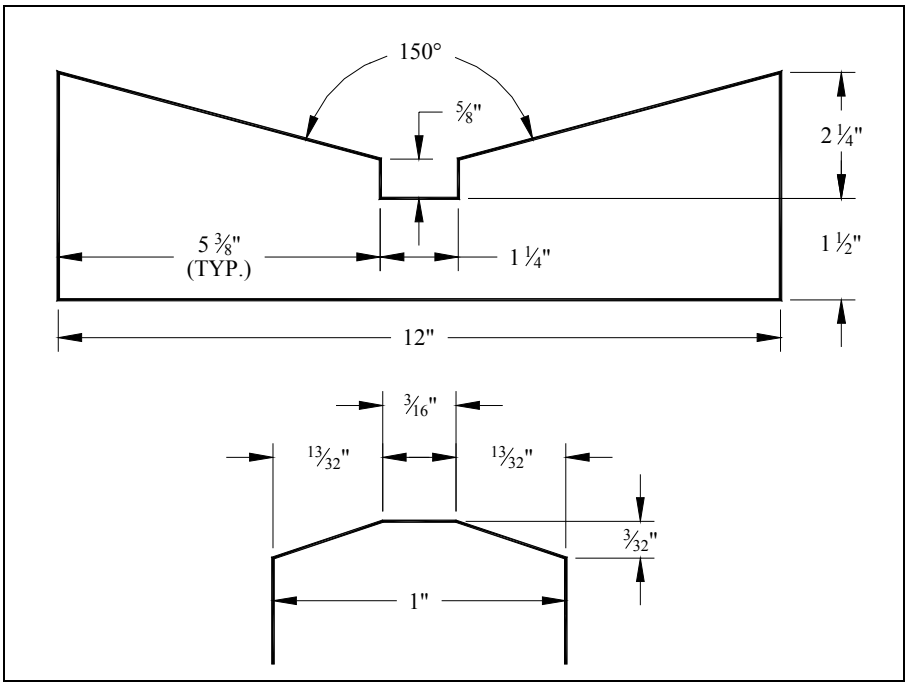

Figure 4.6 – Construction drawing for box culvert weir model.

Both models were fabricated in the Structures and Materials Laboratory by Brett Farmer of the Civil Engineering Department. Figure 4.7 shows both completed models. A wooden strip was later added to the bottom of both models so they could be fastened to the floor of the 1' wide hydraulic flume in the CEE instructional hydraulics laboratory, located in the basement of Patton Hall.

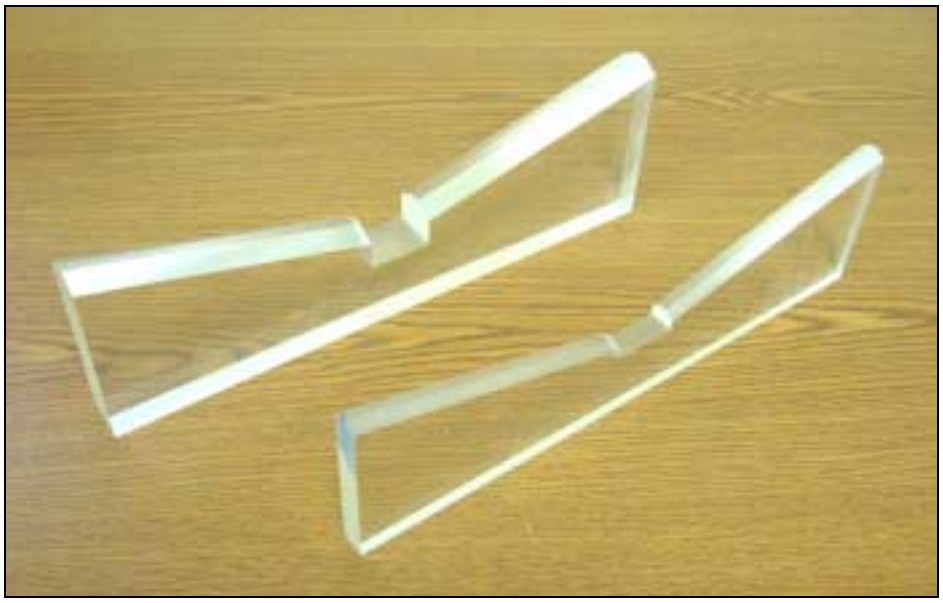

Figure 4.7 – Completed weir models.

4.5.3 WEIR MODEL TESTING AND CALIBRATION

The model weirs were attached to the floor of the hydraulic flume in Patton Hall, and the sides and bottom of the model were sealed to the flume with tape to prevent leakage. The pumps were then started and flow established through the flume at the desired flow rate. At each flow rate, the pump manometer levels and the water surface upstream of the weir were recorded. The hydraulic flume has two built-in pumps. The outflow from each pump is controlled by a valve and orifice, and the pressure difference between the inlet and outlet side of the pump is shown on that pump's manometer tubes. Discharge equations have been developed for the orifices (small and large) that relate the difference in head between the manometers (ΔH) to flow (Q):

$$Q_{small} = 0.057(\Delta H)^{0.5} \qquad [4\text{-}8]$$

$$Q_{large} = 0.2009(\Delta H)^{0.5} \qquad [4\text{-}9]$$

Pump #1 has both small and large orifices available; pump #2 only has a large orifice. Table 4.3 lists the experimental data taken from weir #1 (box culvert weir). Note that the *L* and *R* columns refer to left and right manometer readings, respectively, and *DH* is the difference between them. *WSEL* is the water surface elevation in centimeters. The invert of the weir is at elevation 25.35 cm.

Table 4.3 – Experimental data from weir #1.

WSEL	Pump 1				Pump 2			
	L	R	DH	Q	L	R	DH	Q
cm	ft	ft	ft	cfs	ft	ft	ft	cfs
26.86	1.360	1.365	0.005	0.0040				
27.20	1.355	1.370	0.015	0.0070				
27.95	1.345	1.380	0.035	0.0107				
28.40	1.695	1.780	0.085	0.0166				
28.54	1.690	1.800	0.110	0.0189				
28.69	1.690	1.835	0.145	0.0217				
28.79	1.680	1.860	0.180	0.0242				
28.90	1.660	1.885	0.225	0.0270				
29.12	1.605	1.945	0.340	0.0332				
29.23	1.580	1.990	0.410	0.0365				
29.33	1.540	2.040	0.500	0.0403				
29.50	1.455	2.125	0.670	0.0467				
29.62	1.390	2.195	0.805	0.0511				
29.77	1.270	2.315	1.045	0.0583				
29.89	1.185	2.415	1.230	0.0632				
30.02	1.055	2.550	1.495	0.0697				
Data above are for small orifice; data below are for large orifice.								
28.73	1.950	1.955	0.005	0.0142				
28.97	1.960	1.970	0.010	0.0201				
29.36	1.960	1.975	0.015	0.0246				
29.63	1.955	1.985	0.030	0.0348				
29.86	1.950	1.990	0.040	0.0402				
30.07	1.810	1.875	0.065	0.0512				
30.37	1.785	1.885	0.100	0.0635				
30.58	1.765	1.900	0.135	0.0738				

Table 4.3 (continued) – Experimental data from weir #1.

WSEL	Pump 1				Pump 2			
	L	R	DH	Q	L	R	DH	Q
cm	ft	ft	ft	cfs	ft	ft	ft	cfs
30.76	1.745	1.920	0.175	0.0840				
30.94	1.715	1.935	0.220	0.0942				
31.18	1.660	1.960	0.300	0.1100				
31.45	1.605	1.980	0.375	0.1230				
31.69	1.530	2.005	0.475	0.1385				
31.84	1.455	2.025	0.570	0.1517				
32.00	1.380	2.040	0.660	0.1632				
32.16	1.305	2.055	0.750	0.1740				
32.28	1.215	2.065	0.850	0.1852				
32.44	1.130	2.065	0.935	0.1943				
32.81	1.130	2.065	0.935	0.1943	0.935	0.905	0.030	0.0348
33.03	1.130	2.065	0.935	0.1943	0.950	0.875	0.075	0.0550
33.28	1.130	2.065	0.935	0.1943	0.965	0.830	0.135	0.0738
33.46	1.130	2.065	0.935	0.1943	0.995	0.765	0.230	0.0963
33.69	1.130	2.065	0.935	0.1943	1.035	0.665	0.370	0.1222
34.02	1.130	2.065	0.935	0.1943	1.080	0.500	0.580	0.1530
34.26	1.130	2.065	0.935	0.1943	1.110	0.370	0.740	0.1728
34.37	1.130	2.065	0.935	0.1943	1.345	0.490	0.855	0.1858
34.54	1.130	2.065	0.935	0.1943	1.370	0.365	1.005	0.2014
34.60	1.110	2.095	0.985	0.1994	1.370	0.365	1.005	0.2014
34.88	1.09	2.065	0.975	0.1984	1.385	0.195	1.190	0.2192

The above data were transformed to a depth-discharge rating curve for the prototype weir #1 using equations [4-6] and [4-7]. Table 4.4 shows the tabular data for the prototype, and figure 4.8 shows the data graphically.

Table 4.4 – Rating curve data for prototype weir #1.

Model		Prototype		
Depth *ft*	**Flow** *cfs*	**Depth** *ft*	**Flow** *cfs*	
0.050	0.004	0.46	1.0	Pump #1
0.061	0.007	0.56	1.8	(small orifice)
0.085	0.011	0.79	2.8	
0.100	0.017	0.93	4.3	
0.105	0.019	0.97	4.9	
0.110	0.022	1.01	5.6	
0.113	0.024	1.04	6.3	
0.116	0.027	1.08	7.0	
0.124	0.033	1.14	8.6	
0.127	0.036	1.18	9.5	
0.131	0.040	1.21	10.5	
0.136	0.047	1.26	12.1	
0.140	0.051	1.30	13.3	
0.145	0.058	1.34	15.2	
0.149	0.063	1.38	16.5	
0.153	0.070	1.42	18.1	
0.111	0.014	1.03	3.7	Pump #1
0.119	0.020	1.10	5.2	(large orifice)
0.132	0.025	1.22	6.4	
0.140	0.035	1.30	9.1	
0.148	0.040	1.37	10.5	
0.155	0.051	1.43	13.3	
0.165	0.064	1.52	16.5	
0.172	0.074	1.59	19.2	

Table 4.4 (continued) – Rating curve data for prototype weir #1.

Model		Prototype		
Depth *ft*	**Flow** *cfs*	**Depth** *ft*	**Flow** *cfs*	
0.177	0.084	1.64	21.9	Pump #1
0.183	0.094	1.70	24.5	(large orifice)
0.191	0.110	1.77	28.6	
0.200	0.123	1.85	32.0	
0.208	0.138	1.92	36.0	
0.213	0.152	1.97	39.5	
0.218	0.163	2.02	42.5	
0.223	0.174	2.07	45.3	
0.227	0.185	2.10	48.2	
0.233	0.194	2.15	50.6	
0.245	0.229	2.26	59.6	Pump #1
0.252	0.249	2.33	64.9	and pump #2
0.260	0.268	2.41	69.8	(large orifice)
0.266	0.291	2.46	75.6	
0.274	0.316	2.53	82.4	
0.284	0.347	2.63	90.4	
0.292	0.367	2.70	95.5	
0.296	0.380	2.74	98.9	
0.302	0.396	2.79	103.0	
0.303	0.401	2.81	104.3	
0.313	0.418	2.89	108.7	

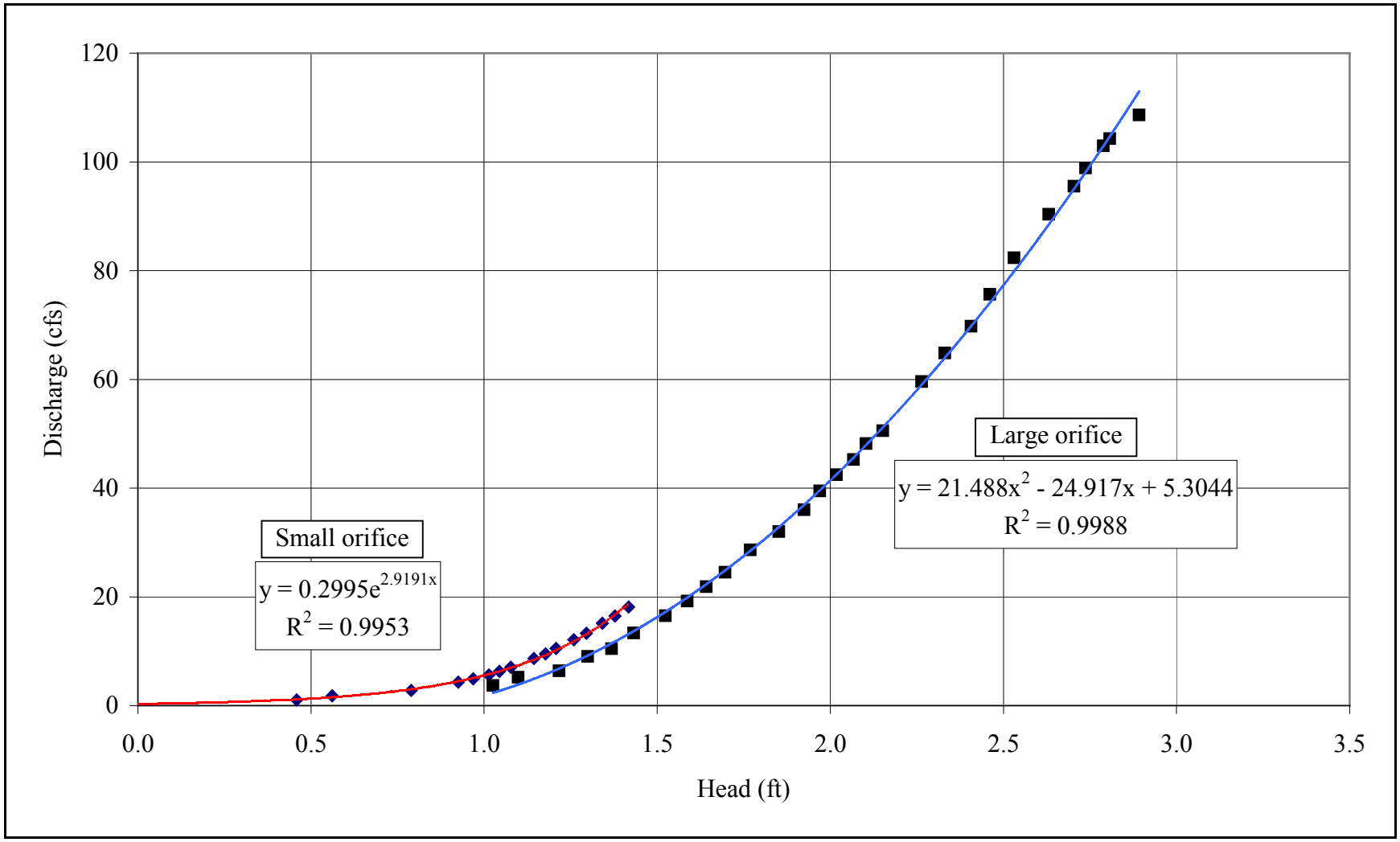

Figure 4.8 – Unadjusted depth-discharge rating curve for prototype weir #1.

Two trend lines have been drawn in figure 4.8. The trend line for the small orifice data points is exponential, and the trend line for the large orifice data points is a second order polynomial. The R^2 values for both lines are very high. However, the two lines do not overlap as would be expected. The reason for the poor match of data is unknown, but is likely due to differences in calibration of the orifices. In order to force a better fit between the two data sets, either the upper curve could be shifted upwards to match the lower curve, or several data points could be eliminated to provide a better fit. For weir #1, the first option (4.5 cfs upward shift) produces a very good fit with a power curve (figure 4.9). For weir #2, however, deleting overlapping data points between the small and large orifice produces a better-fit power curve than does shifting (figure 4.10). Note that both the tabular data and the unadjusted graphical data for weir #2 are provided in Appendix D. Both power curves have exponents near 2.5, which is expected for triangular weirs. Both curves have been projected backward to zero flow and zero head, and forward to maximum depth. Maximum depth for the box culvert (weir #1) is four feet, the height of the culvert. Maximum depth for the footbridge is approximately 4.5 feet. At greater depths, water would flow overland around the ends of the bridge, rendering the rating curve ineffective.

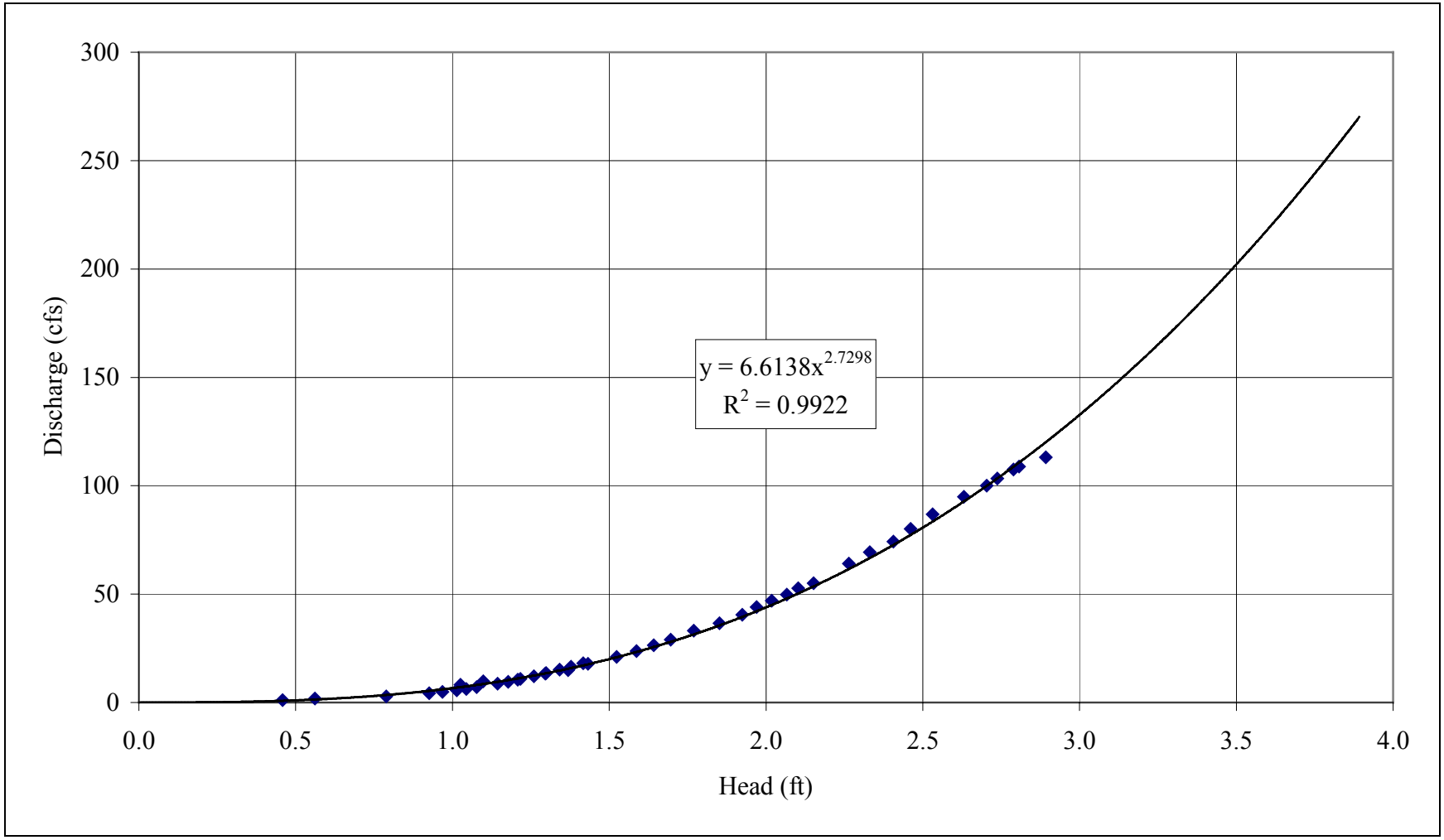

Figure 4.9 – Depth-discharge rating curve for weir #1; large orifice data points shifted upward 4.5 cfs.

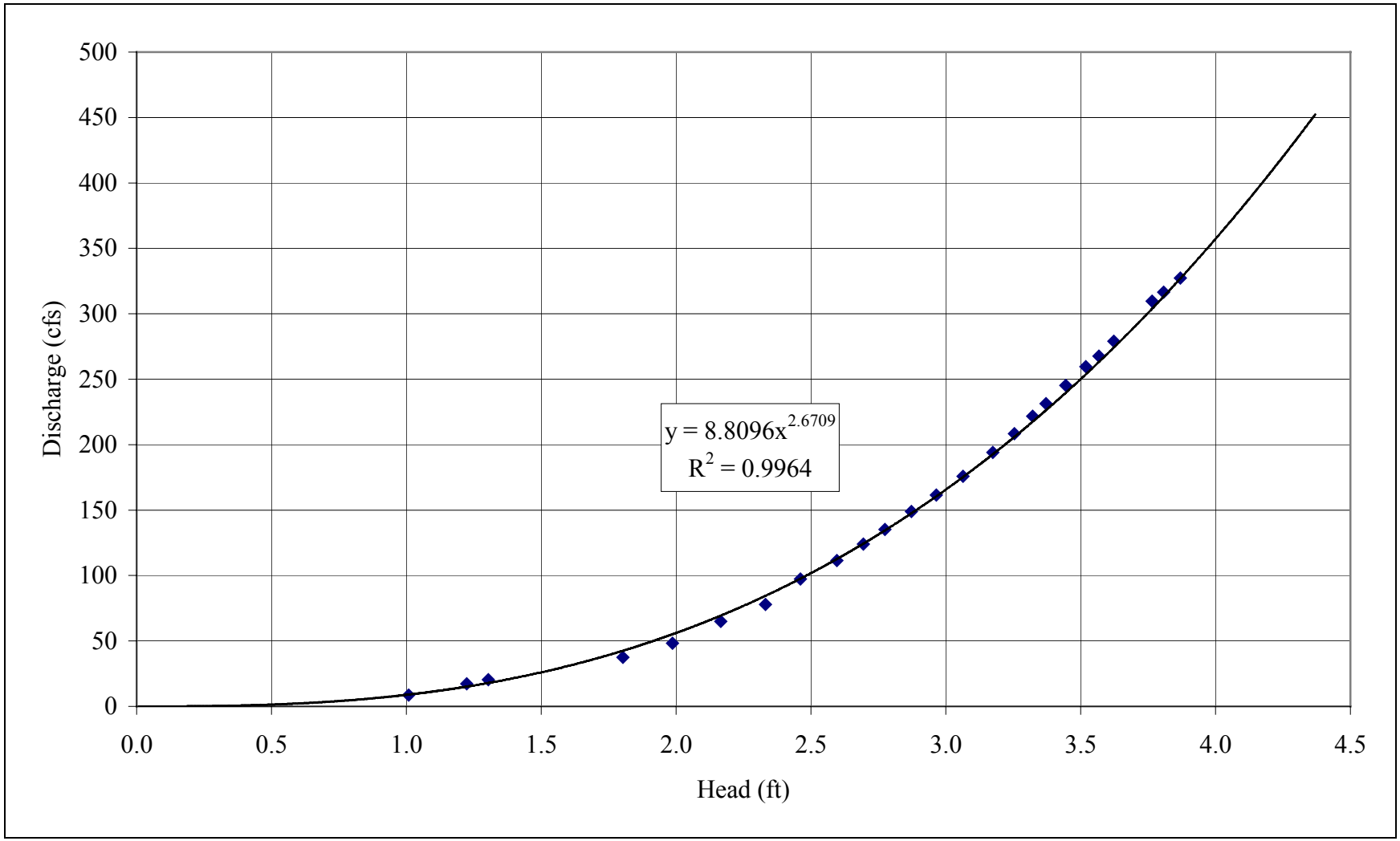

Figure 4.10 – Depth-discharge rating curve for weir #2; overlapping points between large and small orifice deleted from data set.

CHAPTER 5
SUMMARY OF FINDINGS AND RECOMMENDATIONS

5.1 CONSTRUCTION SEQUENCE AND FINAL DESIGNS

Construction for this project will occur in two stages: modifications and improvements to the dam, and filling in the bypass channel. Work on the dam must be completed first before the bypass channel is closed off and flow to the upper pond increased. The first step in any construction project involving earthwork or ground disturbance is installation of erosion control measures. Since this project involves work in live streams and permanent ponds, careful attention must be paid to the installation and maintenance of erosion control measures to avoid severe sediment problems downstream. All erosion control measures will be installed and maintained in accordance with the latest version of the Virginia Erosion and Sediment Control Handbook (VESCH). Construction drawings are included in Appendix E.

The first erosion control measure to be installed will be a construction entrance near the intersection of Duck Pond Drive and West Campus Drive (VESCH Std. & Spec. 3.02). The construction entrance will minimize the amount of mud tracked onto adjacent public streets by construction traffic, and will help minimize onsite disturbance by limiting construction traffic to a single entry and exit point. Safety fence (VESCH Std. & Spec. 3.01) should be installed around the entire construction site to prevent unauthorized access during construction.

5.1.1 DAM MODIFICATIONS

Before work on the dam improvements and modifications can begin, flow into the pond must be diverted to the bypass channel and the pond must be drained. A temporary diversion above the bypass channel will be constructed from riprap faced with sandbags in the location shown on the construction plans. The riprap can be re-used later in construction, and the sandbags will be an inexpensive way to temporarily seal the riprap. Draining the pond can be accomplished with the existing drain valve and pipe in the southwest corner of the pond. If the drain is no longer functional, temporary pumps will

be required to empty the pond. Pumping of sediment-laden water will require the use of a dewatering structure (VESCH Std. & Spec. 3.26). Once flow is diverted into the bypass channel and the pond is drained sufficiently, a type I turbidity curtain (VESCH Std. & Spec. 3.27) must be installed below the upper pond dam. This curtain will minimize the amount of sediment allowed into the lower pond during earthwork activities on the dam. The curtain cannot be installed until there is no longer flow through the existing large staircase structure.

After the pond has been drained and all erosion control measures are in place, earthwork on the dam and low-flow channel may commence. The top of the dam will be stripped of sod and regraded to elevation 2025.10' (see construction drawings in Appendix E for details and areas to be graded). All disturbed areas on the dam will then be covered with a permanent treatment-2 soil stabilization mat (VESCH Std. & Spec. 3.36) and re-seeded. The soil stabilization mat will be stapled using staple pattern D in plate 3.36-6 in the VESCH. All sides of the pond will be protected with riprap along the normal pool water line.

The low-flow channel will be 100' wide at the upper pond, and gradually narrow to 50' where it enters the lower pond (figure 5.1). The invert of the low-flow channel will be at elevation 2023.50' at the upper pond, and will fall to the lower pond at a maximum slope of 5%. The entire width of the low-flow channel will be protected with interlocking concrete block (see construction drawings for details). The concrete blocks will then be backfilled with topsoil and seeded using permanent seeding (VESCH Std. & Spec. 3.32).

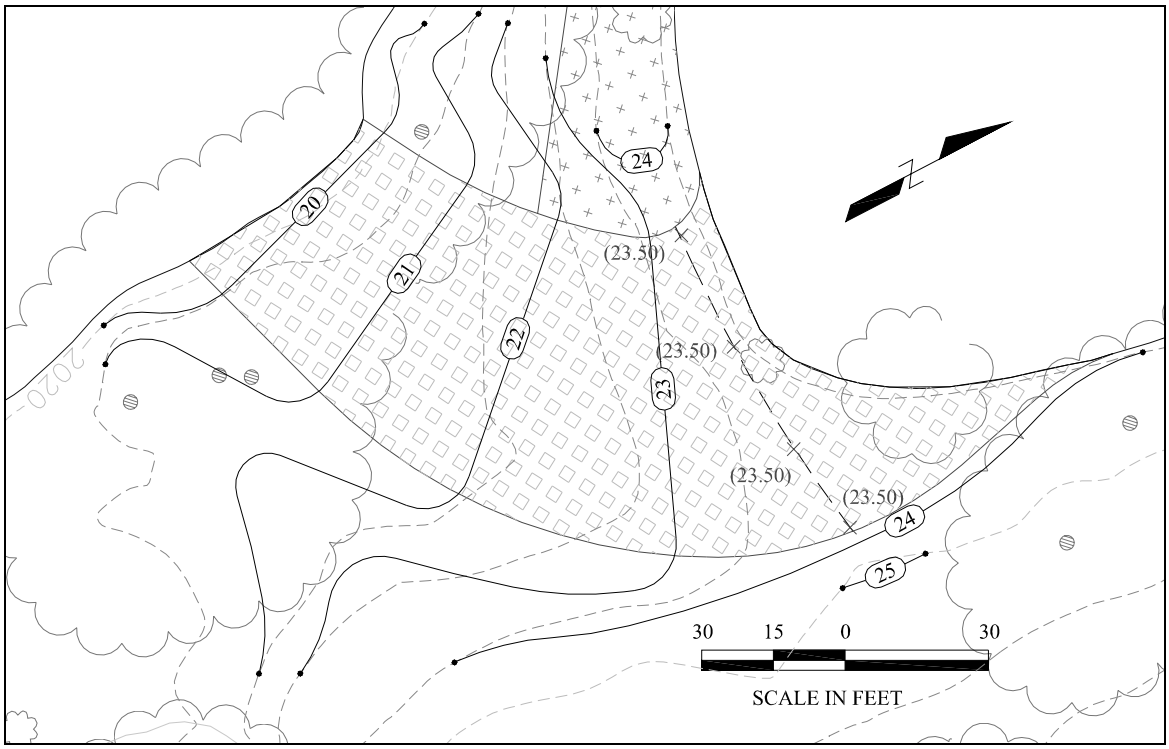

Figure 5.1 – Final plan for low-flow channel.

The turbidity curtain and flow diversion must remain in place until construction on the dam, low-flow channel, and regraded stream bank is complete. Once grass is fully established over the disturbed areas (80% coverage within any 10' x 10' area), the turbidity curtain below the dam may be removed carefully and the temporary diversion may be moved so that all flow is directed into the upper pond. The diversion must be located such that earthwork above the small staircase structure may proceed. Once the temporary diversion is in place, work may begin on the bypass channel.

5.1.2 BYPASS CHANNEL

Construction progress on the proposed Alumni Center will determine how construction on the bypass channel must proceed. The Alumni Center construction includes enlarging the existing water quality pond that collects water from the commuter parking lot on Prices Fork Road. The new, larger water quality pond will accept runoff from the Alumni Center and will outlet into the existing bypass channel along Duck Pond Drive. The outlet pipe from the Alumni Center pond must be extended toward the

footbridge over the bypass channel so that the channel can be filled. The extended outlet pipe will terminate just upstream of the footbridge with a headwall and wingwalls. A drop inlet will be installed at the junction of the Alumni Center outlet and the new extension to capture flow from Duck Pond Drive and the filled bypass channel. Silt fence (VESCH Std. & Spec. 3.05) will be used upstream of the footbridge to prevent silt-laden runoff from entering the lower pond. If installed as part of this project, the drop inlet will be protected with inlet protection (VESCH Std. & Spec. 3.07).

If the Alumni Center pond is under construction while this project is also under construction, the pond outlet pipe may be extended concurrently. If construction on the Center has not yet begun, then the bypass channel should be filled as part of this project and the outlet pipe extended later. Alternate grading plans for the bypass channel are shown in the construction plans for both eventualities.

The small staircase structure at the inlet of the bypass channel will be removed, and an earthen berm built up in its place. The berm will be protected with interlocking concrete blocks similar to the stream bank protection. The bypass channel will be filled, leaving a broad, shallow swale to capture runoff from Duck Pond Drive. This swale will drain to the southwest, toward a drop inlet or into the lower pond, depending on the Alumni Center construction. Once grass has been established on the berm and the filled channel using permanent seeding, the temporary diversion may be removed.

5.1.3 STREAM BANK STABILIZATION

Construction work on stabilizing the banks along both branches of Stroubles Creek may be done concurrently with the pond and bypass channel construction. Careful attention must be paid to erosion control measures on this part of the project to prevent large quantities of silt-laden runoff from entering the streams and ponds. Turbidity curtains would not be appropriate measures, however, due to the shallow depths and currents present in both streams.

Any riprap present along the stream banks must be removed prior to construction. The stream banks will then be regraded and armored with concrete block as shown in figure 4.3. The concrete block will be filled with topsoil and seeded using permanent seeding. The banks must be protected until grass is fully established (80% coverage on

any 10’ x 10’ area), preferably with treatment-1 soil stabilization matting or other semi-structural protective cover. All matting must be anchored per the manufacturer’s specifications.

5.2 DESIGN SELECTION RATIONALE

The final design for modifications to the dam was selected from seven alternative designs because it protects the dam with the least disturbance, cost, and obtrusiveness. There is minimal disturbance to existing vegetation; only minor grading is required across most of the dam, no concrete block or live wells are required around existing trees, and no additional staircase structure, which would have resulted in major grading, is required. Additionally, a second staircase structure would have increased construction costs dramatically. Relatively little concrete block will be required to reinforce the dam for this design alternative, instead utilizing less expensive riprap and geotextile fabric. The design selected also results in minimal depth of flow over the dam; the 100-year storm only results in 0.45’ (approx. 5”) of flow over the dam. The low-flow channel will prevent any dam overtopping for rainfall events equal to and smaller than the 2-year storm.

The stream bank stabilization design will protect the banks from erosion and slumping, and the interlocking concrete blocks will allow grass to grow over them, reducing their visibility. The blocks will not trap floating debris (as the riprap along the stream banks has), will pose less of a safety hazard than riprap, and will require less maintenance.

The 150° V-notch weirs designed for the box culvert and footbridge were selected to minimize the height of the weirs, thus minimizing obstruction to flow in both channels. The rectangular notches in both weirs will allow a greater base flow than V-notch weirs, and will prevent the build-up of sediment behind the weirs that could cause the failure of flow monitoring equipment.

5.3 RECOMMENDATIONS

Once the pond retrofit construction discussed in section 5.1 is complete, more work will be required to analyze the efficiency of the ponds in improving downstream water quality. The primary tools for this analysis will be flow and water quality monitoring equipment. Flow monitoring equipment should be installed at both weirs, and water quality monitoring equipment should be installed upstream and downstream of both ponds. Data from these devices must be analyzed and interpreted to see how much the pond retrofit improves downstream water quality. Based on a similar project, the ponds should remove sediment, nutrients, and fecal coliforms from the water at a rate anywhere from 44% to 97% efficiency, depending on the specific pollutant (City of Austin 2001).

A cursory examination of residence times for the upper pond revealed that the average residence time for the northern branch of Stroubles Creek will not change with removal of the bypass channel. However, the total quantity of water treated in the upper pond will increase significantly. In order to maximize residence time in the upper pond, the pond should be dredged concurrently with this project. If the upper pond is dredged, elevation readings should be taken across the bottom of the pond once dredging operations are complete. These readings could be used to produce a bathymetric map of the pond, which could then be used as a baseline for comparison with future depth readings to determine volume and rate of sedimentation within the pond. This information would be valuable in analyzing the efficiency of the pond in removing suspended solids from the water.

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APPENDIX A
SOIL SURVEY DATA

There are six general soil mapping units within the watershed of the upper pond as shown in figure A.1 (note the Duck Pond in the lower left corner). These mapping units are listed below with a general description of a typical profile for each soil type (Creggar et al. 1985).

Unit 11 – Duffield-Ernest complex: This unit consists of 45% Duffield soils, 35% Ernest soils and 20% other soils. Duffield soils generally consist of a 7" thick surface layer of silt loam over a subsoil of silty clay loam (7 – 37" depth) and sticky plastic clay (37" – 64" depth). Ernest soils generally consist of a 6" thick surface layer of silt loam over a subsoil of silty clay loam (6 – 26" depth) and firm silty clay loam (26" – 50" depth).

Unit 12 – Frederick and Vertrees silt loams: This unit consists of 40% Frederick soils, 35% Vertrees soils and 25% other soils. Frederick silt loam soils generally consist of a 10" thick surface layer of silt loam over a subsoil of clay loam (10" – 18" depth) and sticky plastic clay (18" – 74" depth). Vertrees silt loam soils generally consist of a 10" thick surface layer of silt loam over a subsoil of sticky plastic clay (10" – 66" depth).

Unit 13 – Frederick and Vertrees cherty silt loams: This unit is very similar in composition to unit 12, with slight variations in the typical profiles of each soil. Frederick cherty silt loam soils generally consist of a 10" thick surface layer of cherty silt loam over a subsoil of sticky plastic clay (10" – 74" depth). Vertrees cherty silty loam soils generally consist of a 9" thick surface layer of cherty silt loam over a subsoil of cherty silty clay loam (9" – 21" depth) and sticky plastic clay (21" – 65" depth).

Unit 16 – Groseclose and Poplimento soils: This unit consists of 45% Groseclose soils, 40% Poplimento soils and 15% other soils. Groseclose soils generally consist of a 10" thick surface layer of loam over a subsoil of sticky plastic clay (10" – 39" depth). Poplimento soils generally consist of a 12" thick surface layer of silt loam over a subsoil of sticky plastic clay (12" – 55" depth).

Unit 18 – Groseclose-Urban land complex: This unit contains approximately 50% Groseclose soils (described above), 20% urban land and 30% other soils. Urban land is listed separately since soils in these areas are usually so disturbed that classification is not practical. Permeability, available water capacity, hydraulic conductivity and other soil characteristics are highly variable in urban areas.

Unit 29 – Udorthents and Urban land: This unit consists primarily of urban land where soils are disturbed and highly variable. Classification is not practical in these areas.

Figure A.1 – A portion of map 13 from the Montgomery County Soil Survey (1” = 1350’).

APPENDIX B

STORMWATER MODEL DESCRIPTIONS

In order to calculate flood peaks in ungaged watersheds, several of the methods described below use a development factor to scale up a "natural" or "undeveloped" flood peak. For all such methods used in this project, the "natural" flood peaks for the upper pond watershed were calculated using equations developed by the Virginia division of the United States Geological Survey for natural ungaged watersheds (Miller 1978):

$$Q_2 = 25.2A^{0.83}S^{0.26}RF \qquad \text{[B-1]}$$

$$Q_{10} = 81.3A^{0.78}S^{0.24}RF \qquad \text{[B-2]}$$

$$Q_{25} = 136A^{0.76}S^{0.23}RF \qquad \text{[B-3]}$$

$$Q_{50} = 198A^{0.74}S^{0.22}RF \qquad \text{[B-4]}$$

$$Q_{100} = 269A^{0.73}S^{0.21}RF \qquad \text{[B-5]}$$

where Q_{TR} is the peak flow in cfs for a return period of *TR* years, *A* is the watershed area in square miles, *S* is the main channel slope in feet per mile, and *RF* is a regional factor. The main channel slope is determined between points at 10 and 85 percent of the distance along the main channel from the watershed outlet to the farthest reach of the watershed. For the upper pond, the following values were used:

$$A = \frac{715\ ac}{640\ ac/mi^2} = 1.117\ mi^2 \qquad \text{[B-6]}$$

$$S = \frac{2168' - 2042'}{7821'} \div 5280\ ft/mi = 85.1\ ft/mi \qquad \text{[B-7]}$$

$$RF = 1.1 \qquad \text{[B-8]}$$

Using equations [B-1] through [B-5] gave the following "natural" peak flows for the upper pond:

Q_2 = 96 cfs
Q_{10} = 283 cfs
Q_{25} = 452 cfs
Q_{50} = 628 cfs
Q_{100} = 816 cfs

B.1 BASIN DEVELOPMENT FACTOR (USGS/BDF)

The basin development factor (BDF) method of estimating flood peaks in ungaged watersheds is one of several methods developed by the United States Geological Survey (USGS) and is described in detail in the USGS Water-Supply Paper 2207 (Sauer et al. 1983). In this method, the watershed of interest is divided into three equal segments, beginning at the watershed outlet and radiating upstream. Each of the three segments is then examined for its magnitude of development in four areas: channel improvements, channel linings, storm drains and sewers, and curb-and-gutter streets. For each of these four areas of development, if more than 50% of the item of interest is developed, a value of 1 is assigned. If less than 50% is developed, a value of 0 is assigned. All twelve values are then added together to compute the watershed's BDF. If the entire watershed is developed, then it will score a BDF of 12; an undeveloped watershed will receive a score of 0. Since the watershed draining to the upper pond is nearly 100% developed, it scored a BDF of 11.

The BDF is then used in the following equations to calculate the "urban" peak flows:

$$UQ_2 = 13.2A^{0.21}(13 - BDF)^{-0.43} RQ_2^{0.73} \quad \text{[B-9]}$$

$$UQ_{10} = 9.51A^{0.16}(13 - BDF)^{-0.36} RQ_{10}^{0.79} \quad \text{[B-10]}$$

$$UQ_{25} = 8.68A^{0.15}(13 - BDF)^{-0.34} RQ_{25}^{0.80} \quad \text{[B-11]}$$

$$UQ_{50} = 8.04A^{0.15}(13 - BDF)^{-0.32} RQ_{50}^{0.81} \quad \text{[B-12]}$$

$$UQ_{100} = 7.70A^{0.15}(13 - BDF)^{-0.32} RQ_{100}^{0.82} \quad \text{[B-13]}$$

where UQ_{TR} is the urban peak flow in cfs of return period *TR* years, *A* is the watershed area in square miles, *BDF* is the basin development factor, and RQ_{TR} is the "rural" or "natural" peak flow in cfs of return period *TR* years.

B.2 STANKOWSKI RATIO METHOD

One method developed by S. J. Stankowski of the USGS in 1974 (Stankowski 1974) uses a simple ratio of rural to urban runoff based on the percent manmade impervious cover in the watershed. Table B.1 lists Stankowski's ratios for given impervious cover percentages.

Table B.1 – Stankowski runoff ratios based on impervious fraction (Stankowski 1974).

Recurrence Interval (years)	Percent Manmade Impervious Cover				
	1	**10**	**25**	**50**	**80**
2	1.0	1.8	2.2	2.6	3.0
5	1.0	1.6	2.0	2.4	2.6
10	1.0	1.6	1.9	2.2	2.4
25	1.0	1.5	1.8	2.0	2.2
50	1.0	1.4	1.7	1.9	2.0
100	1.0	1.4	1.6	1.7	1.8

The watershed draining to the upper pond has an average of 31.9% impervious cover. Using linear interpolation in the table above, the following ratios were computed:

Table B.2 – Stankowski ratios for the upper pond watershed.

Recurrence Interval (yrs)	Stankowski Ratio
2	2.3
10	1.95
25	1.85
50	1.75
100	1.63

B.3 STANKOWSKI REGRESSION EQUATIONS

A second method developed by Stankowski for the USGS (Stankowski 1974) uses regression equations:

$$Q_2 = 25.6A^{0.89}S^{0.25}St^{-0.56}I^{0.25} \quad \text{[B-14]}$$

$$Q_{10} = 54.0A^{0.88}S^{0.27}St^{-0.53}I^{0.20} \quad \text{[B-15]}$$

$$Q_{25} = 78.2A^{0.86}S^{0.27}St^{-0.52}I^{0.18} \quad \text{[B-16]}$$

$$Q_{50} = 104A^{0.85}S^{0.26}St^{-0.51}I^{0.16} \quad \text{[B-17]}$$

$$Q_{100} = 136A^{0.84}S^{0.26}St^{-0.51}I^{0.14} \quad \text{[B-18]}$$

where Q_{TR} is the flood peak in cfs for a return period of *TR* years, *A* is the watershed area in square miles, *S* is the main channel slope in feet per mile, calculated as in equation [B-7], *St* is the fraction of the watershed covered by lakes or swamps plus 1%, and *I* is the fraction of manmade impervious cover in the watershed. For the upper pond:

$A = 1.117$ mi^2 $S = 85.1$ ft/mi $St = 0.01$ $I = 0.319$

B.4 RATIONAL METHOD

The rational method is widely used to calculate peak flows in small, ungaged watersheds. The equation for the rational method is:

$$Q_{TR} = Ci_{TR}A \quad \text{[B-19]}$$

where Q_{TR} is the peak flow in cfs for a return period of *TR* years, *C* is a dimensionless runoff coefficient based on land cover (often called the "C-factor"), i_{TR} is the average rainfall intensity in inches per hour for a storm of return period *TR* years, and *A* is the watershed drainage area in acres. The rainfall intensity is calculated using an intensity-duration-frequency chart (figure B.1) and the time of concentration, T_c, for the watershed. The time of concentration may be calculated several ways, but for this project, an assumed time of concentration of 1 hour was used. The equivalent rainfall intensities for a time of concentration of 1 hour are shown in figure B.1. Using a C-factor of 0.9 for impervious areas and a factor of 0.3 for pervious areas, the upper pond watershed has an overall C-factor of 0.50.

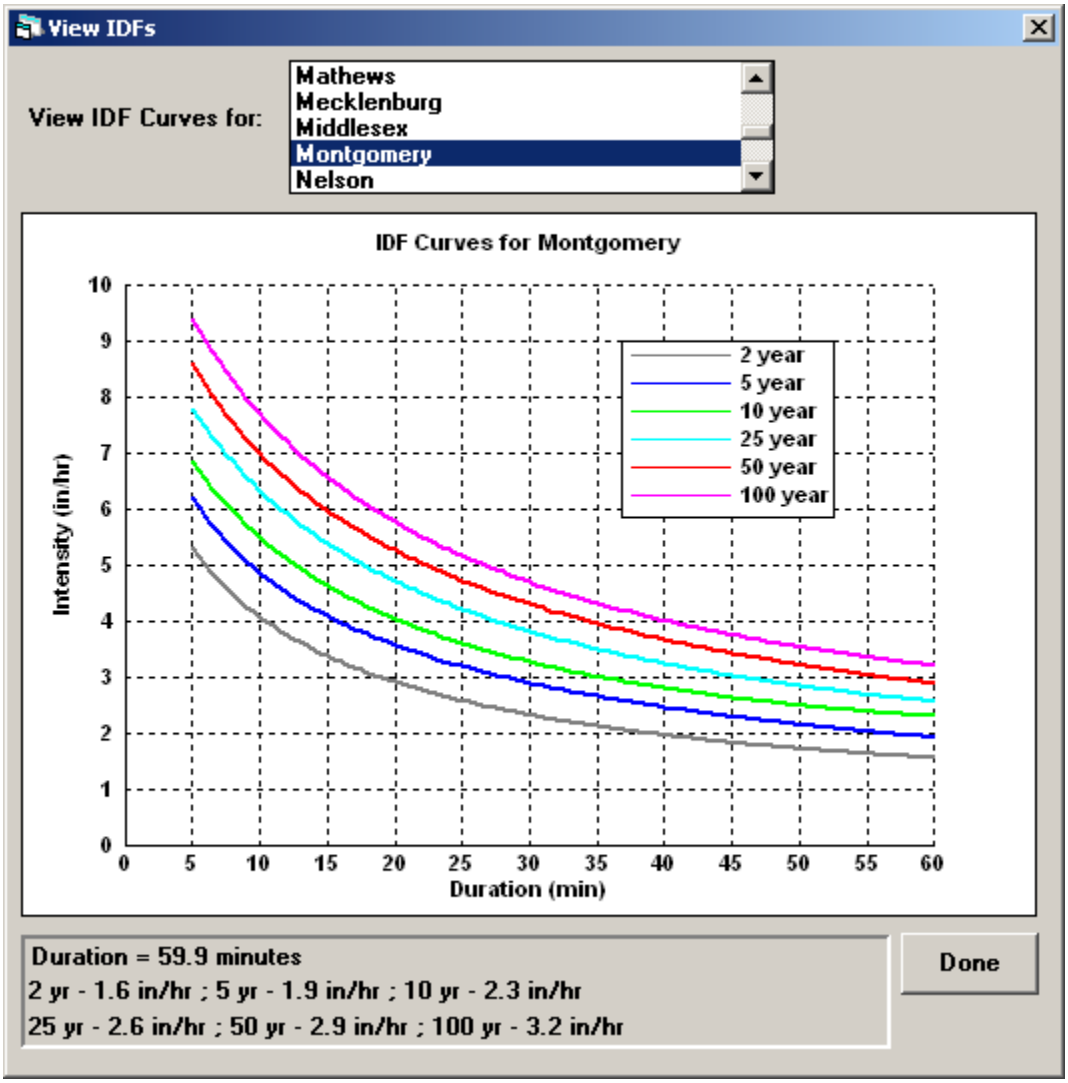

Figure B.1 – Intensity-duration-frequency (IDF) chart for Montgomery County, Virginia[2].

B.5 SCS TECHNICAL REPORT 55 (TR-55)

Technical report #55 from the Soil Conservation Service, "Urban Hydrology for Small Watersheds" (USDA NRCS 1986) describes a method of developing hydrographs for ungaged watersheds. This method is often referred to as TR-55, and is widely used for its simplicity and capacity to model multiple subwatersheds. Based on time of concentration, travel time and curve number, unit discharges ($ft^3/sec/mi.^2/in.$) are selected from tables developed by the SCS for type II rainfall distributions. The unit discharges

[2] Screen shot of the Montgomery County IDF chart was taken from VTPSUHM.

are then scaled based on the area and rainfall in each subwatershed, and the subwatershed discharges are added together at each time step to produce a total hydrograph.

VTPSUHM was used to create hydrographs using the TR-55 tabular method. The required inputs are the 24-hour rainfall depth across the watershed, and the area, curve number, time of concentration, and travel time for each subwatershed. For TR-55, travel time is defined as the flow time from a subwatershed outlet to the point of interest. The 24-hour rainfall depth for each storm was taken from the hyetographs used in the EPA-SWMM model. Curve numbers for each subwatershed were calculated using the area-weighting method described in section 2.6. Times of concentration were calculated using the kinematic wave method in VTPSUHM. Travel times were computed using design velocities from the EPA-SWMM model. Table B.3 lists the input parameters for each subwatershed. Output from VTPSUHM is included in Appendix C.

Table B.3 – TR-55 input parameters.

Subarea	Area (ac)	Curve Number	Time of Concentration (min.)	Travel Time (min.)
1	39.7	84	28.4	38.8
2	71.0	79	35.0	23.8
3	50.0	77	32.6	18.1
4	60.9	77	30.1	17.1
5	35.3	80	27.7	17.4
6	65.7	81	21.2	9.4
7	50.8	84	25.5	13.7
8	39.9	78	26.2	10.3
9	49.0	79	43.1	8.3
10	94.4	84	22.4	5.0
11	63.6	79	47.2	3.8
12	95.1	90	42.6	0.0

APPENDIX C
VTPSUHM MODELING OUTPUT

C.1 DAM MODIFICATION CALCULATIONS

Following is the output from VTPSUHM for the testing of the final dam modification design. The 2-, 10-, and 100-year storms were routed through the upper pond to check peak water surface elevations and depth of flow over the dam. Also included are the sizing calculations for the low-flow channel as it enters the lower pond.

Basin Storage/Elevation Input

	Elevation (ft)	Storage (acre-ft)
Normal pool elevation →	2023.4	0
	2024	1.265
	2024.5	2.134
	2025	3.109
	2025.5	4.2
	2027	7.1

Note: Top of proposed dam for design alternative #2 is 2025.10'

Outlet Structure Configuration

Outlet Structure Configuration for Design Alternative #2

Stage 1: Rectangular Weir
Crest Elevation = 2023.27 feet
Length = 10.8 feet
Discharge Coefficient = 3.1

Note: Stages 1, 2, and 5 are parts of the existing staircase structure

Stage 2: Rectangular Weir
Crest Elevation = 2023.6 feet
Length = 20.6 feet
Discharge Coefficient = 3.1

Stage 3: Rectangular Weir
Crest Elevation = 2023.5 feet
Length = 100 feet
Discharge Coefficient = 3.1
} Low flow channel

Stage 4: Rectangular Weir
Crest Elevation = 2025.1 feet
Length = 200 feet
Discharge Coefficient = 3.1
} Top of dam

Stage 5: Rectangular Weir
Crest Elevation = 2024.14 feet
Length = 25.3 feet
Discharge Coefficient = 3.1

Basin Rating Curve

Basin Water Elevation	Basin Outflow (cfs)	Riser Box Water Elevation	Tailwater Elevation (ft)	Outfall Culvert Control	Outfall Culvert Override?
2023.27	0.00	N/A	N/A	N/A	N/A
2023.32	0.41	N/A	N/A	N/A	N/A
2023.37	1.11	N/A	N/A	N/A	N/A
2023.42	2.01	N/A	N/A	N/A	N/A
2023.47	3.07	N/A	N/A	N/A	N/A
2023.52	5.37	N/A	N/A	N/A	N/A
2023.57	11.75	N/A	N/A	N/A	N/A
2023.62	20.70	N/A	N/A	N/A	N/A
2023.67	32.24	N/A	N/A	N/A	N/A
2023.72	45.75	N/A	N/A	N/A	N/A
2023.77	60.93	N/A	N/A	N/A	N/A
2023.82	77.61	N/A	N/A	N/A	N/A
2023.87	95.65	N/A	N/A	N/A	N/A
2023.92	114.96	N/A	N/A	N/A	N/A
2023.97	135.45	N/A	N/A	N/A	N/A
2024.02	157.06	N/A	N/A	N/A	N/A
2024.07	179.73	N/A	N/A	N/A	N/A
2024.12	203.42	N/A	N/A	N/A	N/A
2024.17	228.57	N/A	N/A	N/A	N/A
2024.22	255.58	N/A	N/A	N/A	N/A
2024.27	284.01	N/A	N/A	N/A	N/A
2024.32	313.72	N/A	N/A	N/A	N/A
2024.37	344.62	N/A	N/A	N/A	N/A
2024.42	376.65	N/A	N/A	N/A	N/A
2024.47	409.75	N/A	N/A	N/A	N/A
2024.52	443.88	N/A	N/A	N/A	N/A

Basin Water Elevation	Basin Outflow (cfs)	Riser Box Water Elevation	Tailwater Elevation (ft)	Outfall Culvert Control	Outfall Culvert Override?
2024.57	479.00	N/A	N/A	N/A	N/A
2024.62	515.09	N/A	N/A	N/A	N/A
2024.67	552.11	N/A	N/A	N/A	N/A
2024.72	590.04	N/A	N/A	N/A	N/A
2024.77	628.86	N/A	N/A	N/A	N/A
2024.82	668.54	N/A	N/A	N/A	N/A
2024.87	709.06	N/A	N/A	N/A	N/A
2024.92	750.40	N/A	N/A	N/A	N/A
2024.97	792.56	N/A	N/A	N/A	N/A
2025.02	835.50	N/A	N/A	N/A	N/A
2025.07	879.22	N/A	N/A	N/A	N/A
2025.12	926.13	N/A	N/A	N/A	N/A
2025.17	981.65	N/A	N/A	N/A	N/A
2025.22	1042.29	N/A	N/A	N/A	N/A
2025.27	1106.99	N/A	N/A	N/A	N/A
2025.32	1175.19	N/A	N/A	N/A	N/A
2025.37	1246.57	N/A	N/A	N/A	N/A
2025.42	1320.86	N/A	N/A	N/A	N/A
2025.47	1397.88	N/A	N/A	N/A	N/A
2025.52	1477.47	N/A	N/A	N/A	N/A
2025.57	1559.52	N/A	N/A	N/A	N/A
2025.62	1643.91	N/A	N/A	N/A	N/A
2025.67	1730.55	N/A	N/A	N/A	N/A
2025.72	1819.37	N/A	N/A	N/A	N/A
2025.77	1910.29	N/A	N/A	N/A	N/A
2025.82	2003.25	N/A	N/A	N/A	N/A
2025.87	2098.20	N/A	N/A	N/A	N/A
2025.92	2195.08	N/A	N/A	N/A	N/A
2025.97	2293.84	N/A	N/A	N/A	N/A
2026.02	2394.44	N/A	N/A	N/A	N/A
2026.07	2496.84	N/A	N/A	N/A	N/A
2026.12	2601.00	N/A	N/A	N/A	N/A
2026.17	2706.89	N/A	N/A	N/A	N/A
2026.22	2814.47	N/A	N/A	N/A	N/A
2026.27	2923.72	N/A	N/A	N/A	N/A
2026.32	3034.60	N/A	N/A	N/A	N/A
2026.37	3147.09	N/A	N/A	N/A	N/A
2026.42	3261.16	N/A	N/A	N/A	N/A
2026.47	3376.79	N/A	N/A	N/A	N/A

Modified Puls Routing

Inflow Hydrograph: F/SWMM-2.hyd
Storage/Elevation Curve: F/Pond1.ES
Discharge/Elevation Curve: F/Pond1.EO

2-year storm routing

Basin Bypass Capacity = 0.0 cfs
Starting Pool Elevation = 2023.40 feet ← Normal pool elevation
Time Interval = 4.166667E-02 hours

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
11.00	27.92	27.92	0.000	2023.40	0.00	0.00
11.04	28.54	28.54	0.093	2023.44	2.52	2.52
11.08	29.47	29.47	0.182	2023.49	3.82	3.82
11.13	30.38	30.38	0.268	2023.53	6.25	6.25
11.17	31.29	31.29	0.344	2023.56	10.89	10.89
11.21	32.21	32.21	0.407	2023.59	15.91	15.91
11.25	33.10	33.10	0.458	2023.62	20.18	20.18
11.29	33.98	33.98	0.497	2023.64	24.29	24.29
11.33	35.09	35.09	0.526	2023.65	27.54	27.54
11.38	36.10	36.10	0.550	2023.66	30.10	30.10
11.42	36.95	36.95	0.568	2023.67	32.14	32.14
11.46	37.63	37.63	0.583	2023.68	33.98	33.98
11.50	38.18	38.18	0.594	2023.68	35.40	35.40
11.54	39.59	39.59	0.604	2023.69	36.66	36.66
11.58	41.72	41.72	0.615	2023.69	38.10	38.10
11.63	43.91	43.91	0.628	2023.70	39.81	39.81
11.67	46.26	46.26	0.643	2023.71	41.72	41.72
11.71	48.49	48.49	0.659	2023.71	43.76	43.76
11.75	50.75	50.75	0.676	2023.72	45.89	45.89
11.79	52.79	52.79	0.692	2023.73	48.23	48.23
11.83	54.67	54.67	0.707	2023.74	50.41	50.41
11.88	56.33	56.33	0.721	2023.74	52.44	52.44
11.92	57.74	57.74	0.734	2023.75	54.26	54.26
11.96	59.06	59.06	0.745	2023.75	55.91	55.91
12.00	60.17	60.17	0.755	2023.76	57.38	57.38
12.04	90.89	90.89	0.805	2023.78	64.88	64.88
12.08	150.70	150.70	0.955	2023.85	89.53	89.53
12.13	214.37	214.37	1.198	2023.97	134.66	134.66
12.17	291.34	291.34	1.482	2024.13	205.88	205.88
12.21	382.17	382.17	1.775	2024.29	297.78	297.78
12.25	466.73	466.73	2.043	2024.45	395.04	395.04
12.29	538.99	538.99	2.270	2024.57	478.88	478.88
12.33	609.01	609.01	2.469	2024.67	553.54	553.54
12.38	685.89	685.89	2.662	2024.77	629.43	629.43
12.42	759.53	759.53	2.850	2024.87	706.76	706.76
12.46	842.27	842.27	3.037	2024.96	786.59	786.59

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)	
12.50	922.23	922.23	3.229	2025.06	866.15	866.15	
12.54	909.52	909.52	3.328	2025.10	907.91	907.91	Max. WSEL
12.58	857.73	857.73	3.280	2025.08	887.25	887.25	
12.63	767.10	767.10	3.129	2025.01	825.54	825.54	
12.67	664.31	664.31	2.911	2024.90	732.53	732.53	
12.71	599.28	599.28	2.709	2024.80	648.47	648.47	
12.75	548.75	548.75	2.557	2024.72	587.69	587.69	
12.79	513.50	513.50	2.440	2024.66	542.45	542.45	
12.83	482.71	482.71	2.348	2024.61	507.51	507.51	
12.88	456.99	456.99	2.268	2024.57	478.21	478.21	
12.92	434.97	434.97	2.200	2024.53	453.53	453.53	
12.96	415.73	415.73	2.140	2024.50	431.67	431.67	
13.00	398.21	398.21	2.089	2024.47	412.30	412.30	
13.04	378.64	378.64	2.039	2024.45	393.42	393.42	
13.08	356.83	356.83	1.985	2024.42	373.14	373.14	
13.13	334.17	334.17	1.927	2024.38	351.68	351.68	
13.17	311.33	311.33	1.866	2024.35	329.55	329.55	
13.21	290.33	290.33	1.804	2024.31	307.88	307.88	
13.25	273.04	273.04	1.747	2024.28	288.47	288.47	
13.29	258.35	258.35	1.697	2024.25	271.93	271.93	
13.33	244.68	244.68	1.652	2024.22	257.22	257.22	
13.38	232.30	232.30	1.610	2024.20	244.11	244.11	
13.42	220.46	220.46	1.571	2024.18	231.75	231.75	
13.46	210.69	210.69	1.534	2024.16	220.84	220.84	
13.50	202.36	202.36	1.501	2024.14	211.32	211.32	
13.54	194.11	194.11	1.471	2024.12	202.65	202.65	
13.58	185.43	185.43	1.441	2024.10	194.42	194.42	
13.63	177.13	177.13	1.410	2024.08	186.03	186.03	
13.67	170.00	170.00	1.380	2024.07	178.12	178.12	
13.71	162.66	162.66	1.352	2024.05	170.81	170.81	
13.75	155.20	155.20	1.324	2024.03	163.44	163.44	
13.79	148.59	148.59	1.297	2024.02	156.37	156.37	
13.83	142.50	142.50	1.270	2024.00	150.41	150.41	
13.88	136.83	136.83	1.243	2023.99	144.50	144.50	
13.92	131.70	131.70	1.217	2023.98	138.87	138.87	
13.96	127.03	127.03	1.193	2023.97	133.80	133.80	
14.00	122.68	122.68	1.170	2023.96	129.31	129.31	
14.04	118.37	1 18.37	1.148	2023.94	124.91	124.91	
14.08	113.87	113.87	1.125	2023.93	120.50	120.50	
14.13	109.46	109.46	1.102	2023.92	116.07	116.07	
14.17	105.14	105.14	1.079	2023.91	111.82	111.82	
14.21	100.69	100.69	1.056	2023.90	107.55	107.55	
14.25	96.75	96.75	1.033	2023.89	103.32	103.32	
14.29	93.04	93.04	1.011	2923.88	99.28	99.28	
14.33	89.59	89.59	0.990	2023.87	95.47	95.47	
14.38	86.34	86.34	0.970	2023.86	92.05	92.05	
14.42	83.35	83.35	0.951	2023.85	88.77	88.77	

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
14.46	80.56	80.56	0.933	2023.84	85.67	85.67
14.50	77.99	77.99	0.916	2023.83	82.76	82.76
14.54	75.25	75.25	0.899	2023.83	79.96	79.96
14.58	72.36	72.36	0.883	2023.82	77.19	77.19
14.63	69.47	69.47	0.866	2023.81	74.50	74.50
14.67	66.56	66.56	0.848	2023.80	71.72	71.72
14.71	63.75	63.75	0.831	2023.79	68.91	68.91
14.75	61.03	61.03	0.813	2023.79	66.12	66.12
14.79	58.45	58.45	0.796	2023.78	63.39	63.39
14.83	55.86	55.86	0.779	2023.77	60.73	60.73
14.88	53.62	53.62	0.762	2023.76	58.35	58.35
14.92	51.66	51.66	0.746	2023.75	56.08	56.08
14.96	49.77	49.77	0.732	2023.75	53.95	53.95
15.00	48.07	48.07	0.718	2023.74	51.95	51.95
15.04	46.58	46.58	0.705	2023.73	50.11	50.11
15.08	45.13	45.13	0.693	2023.73	48.42	48.42
15.13	43.75	43.75	0.682	2023.72	46.84	46.84
15.17	42.41	42.41	0.672	2023.72	45.38	45.38
15.21	41.17	41.17	0.662	2023.71	44.08	44.08
15.25	39.99	39.99	0.652	2023.71	42.82	42.82
15.29	38.89	38.89	0.642	2023.71	41.60	41.60
15.33	37.86	37.86	0.633	2023.70	40.43	40.43
15.38	36.89	36.89	0.625	2023.70	39.33	39.33
15.42	36.00	36.00	0.616	2023.69	38.29	38.29
15.46	35.19	35.19	0.609	2023.69	37.31	37.31
15.50	34.41	34.41	0.602	2023.69	36.40	36.40
15.54	33.69	33.69	0.595	2023.68	35.55	35.55
15.58	32.99	32.99	0.589	2023.68	34.75	34.75
15.63	32.35	32.35	0.583	2023.68	34.00	34.00
15.67	31.69	31.69	0.577	2023.67	33.28	33.28
15.71	31.08	31.08	0.572	2023.67	32.60	32.60
15.75	30.51	30.51	0.567	2023.67	31.98	31.98
15.79	29.94	29.94	0.562	2023.67	31.42	31.42
15.83	29.41	29.41	0.557	2023.66	30.87	30.87
15.88	28.93	28.93	0.552	2023.66	30.33	30.33
15.92	28.50	28.50	0.547	2023.66	29.82	29.82
15.96	28.20	28.20	0.543	2023.66	29.35	29.35
16.00	27.89	27.89	0.539	2023.66	28.94	28.94
16.04	27.60	27.60	0.536	2023.65	28.56	28.56
16.08	27.27	27.27	0.532	2023.65	28.20	28.20
16.13	26.96	26.96	0.529	2023.65	27.86	27.86
16.17	26.64	26.64	0.526	2023.65	27.52	27.52
16.21	26.32	26.32	0.523	2023.65	27.19	27.19
16.25	26.05	26.05	0.520	2023.65	26.87	26.87
16.29	25.76	25.76	0.517	2023.65	26.57	26.57
16.33	25.49	25.49	0.515	2023.64	26.27	26.27
16.38	25.19	25.19	0.512	2023.64	25.97	25.97

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
16.42	24.95	24.95	0.509	2023.64	25.69	25.69
16.46	24.70	24.70	0.507	2023.64	25.41	25.41
16.50	24.46	24.46	0.504	2023.64	25.15	25.15
16.54	24.02	24.02	0.502	2023.64	24.86	24.86
16.58	23.37	23.37	0.498	2023.64	24.49	24.49
16.63	22.76	22.76	0.494	2023.64	24.04	24.04
16.67	22.09	22.09	0.490	2023.63	23.53	23.53
16.71	21.43	21.43	0.485	2023.63	22.97	22.97
16.75	20.75	20.75	0.479	2023.63	22.37	22.37
16.79	20.09	20.09	0.473	2023.63	21.75	21.75
16.83	19.28	19.28	0.467	2023.62	21.10	21.10
16.88	18.70	18.70	0.461	2023.62	20.48	20.48
16.92	18.13	18.13	0.455	2023.62	19.95	19.95
16.96	17.63	17.63	0.449	2023.61	19.43	19.43
17.00	17.13	17.13	0.443	2023.61	18.90	18.90
17.04	16.50	16.50	0.436	2023.61	18.37	18.37
17.08	15.94	15.94	0.430	2023.60	17.82	17.82
17.13	15.42	15.42	0.424	2023.60	17.28	17.28
17.17	14.99	14.99	0.417	2023.60	16.75	16.75
17.21	14.62	14.62	0.411	2023.60	16.25	16.25
17.25	0.00	0.00	0.385	2023.58	13.97	13.97

Total Routing Mass Balance Discrepancy is -0.03%

Modified Puls Routing

Inflow Hydrograph: F/SWMM-10.hyd
Storage/Elevation Curve: F/Pond1.ES
Discharge/Elevation Curve: F/Pond1.EO

10-year storm routing

Basin Bypass Capacity = 0.0 cfs
Starting Pool Elevation = 2023.40 feet
Time Interval = 4.166667E-02 hours

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
11.00	49.36	49.36	0.000	2023.40	0.00	0.00
11.04	50.46	50.46	0.166	2023.48	3.47	3.47
11.08	52.11	52.11	0.320	2023.55	9.44	9.44
11.13	53.77	53.77	0.452	2023.62	19.72	19.72
11.17	55.57	55.57	0.554	2023.66	30.57	30.57
11.21	57.22	57.22	0.627	2023.70	39.67	39.67
11.25	58.92	58.92	0.679	2023.72	46.38	46.38
11.29	60.61	60.61	0.716	2023.74	51.70	51.70
11.33	62.13	62.13	0.743	2023.75	55.54	55.54
11.38	63.36	63.36	0.763	2023.76	58.40	58.40
11.42	64.35	64.35	0.778	2023.77	60.57	60.57
11.46	65.10	65.10	0.789	2023.77	62.32	62.32
11.50	65.70	65.70	0.797	2023.78	63.64	63.64
11.54	68.00	68.00	0.806	2023.78	65.01	65.01
11.58	72.12	72.12	0.820	2023.79	67.17	67.17
11.63	76.43	76.43	0.839	2023.80	70.21	70.21
11.67	81.13	81.13	0.862	2023.81	73.88	73.88
11.71	85.54	85.54	0.888	2023.82	77.95	77.95
11.75	89.82	89.82	0.913	2023.83	82.38	82.38
11.79	93.38	93.38	0.938	2023.85	86.58	86.58
11.83	96.34	96.34	0.960	2023.86	90.35	90.35
11.88	98.71	98.71	0.979	2023.86	93.61	93.61
11.92	100.53	100.53	0.995	2023.87	96.39	96.39
11.96	101.86	101.86	1.008	2023.88	98.69	98.69
12.00	102.84	102.84	1.017	2023.88	100.45	100.45
12.04	166.50	166.50	1.107	2023.93	116.94	116.94
12.08	287.38	287.38	1.380	2024.07	178.03	178.03
12.13	450.66	450.66	1.811	2024.31	310.07	310.07
12.17	664.97	664.97	2.333	2024.60	502.11	502.11
12.21	888.26	888.26	2.894	2024.89	725.33	725.33
12.25	1070.42	1070.42	3.399	2025.13	940.23	940.23
12.29	1165.17	1165.17	3.710	2025.28	1114.43	1114.43
12.33	1234.43	1234.43	3.851	2025.34	1203.55	1203.55
12.38	1340.88	1340.88	3.985	2025.40	1293.52	1293.52
12.42	1361.73	1361.73	4.076	2025.44	1356.47	1356.47
12.46	1369.43	1369.43	4.090	2025.45	1366.47	1366.47

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)	
12.50	1376.22	1376.22	4.100	2025.45	1373.44	1373.44	Max. WSEL
12.54	1293.27	1293.27	4.040	2025.43	1330.98	1330.98	
12.58	1178.83	1178.83	3.889	2025.36	1228.69	1228.69	
12.63	1121.86	1121.86	3.761	2025.30	1146.32	1146.32	
12.67	1085.00	1085.00	3.690	2025.27	1101.98	1101.98	
12.71	1062.56	1062.56	3.642	2025.24	1073.49	1073.49	
12.75	1005.54	1005.54	3.574	2025.21	1033.92	1033.92	
12.79	981.89	981.89	3.503	2025.18	994.60	994.60	
12.83	958.15	958.15	3.459	2025.16	971.06	971.06	
12.88	931.17	931.17	3.411	2025.14	946.40	946.40	
12.92	901.73	901.73	3.355	2025.11	919.12	919.12	
12.96	871.42	871.42	3.290	2025.08	891.43	891.43	
13.00	839.83	839.83	3.218	2025.05	861.70	861.70	
13.04	792.31	792.31	3.126	2025.01	824.12	824.12	
13.08	652.67	652.67	2.924	2024.91	738.03	738.03	
13.13	581.81	581.81	2.681	2024.78	637.32	637.32	
13.17	531.66	531.66	2.516	2024.70	571.92	571.92	
13.21	494.61	494.61	2.395	2024.63	525.15	525.15	
13.25	461.99	461.99	2.296	2024.58	488.52	488.52	
13.29	435.15	435.15	2.212	2024.54	457.78	457.78	
13.33	411.50	411.50	2.139	2024.50	431.10	431.10	
13.38	389.85	389.85	2.075	2024.47	407.22	407.22	
13.42	371.08	371.08	2.020	2024.43	386.03	386.03	
13.46	354.76	354.76	1.971	2024.41	367.90	367.90	
13.50	340.30	340.30	1.928	2024.38	352.08	352.08	
13.54	325.32	325.32	1.887	2024.36	337.28	337.28	
13.58	309.47	309.47	1.845	2024.33	322.18	322.18	
13.63	293.41	293.41	1.800	2024.31	306.60	306.60	
13.67	278.54	278.54	1.756	2024.28	291.32	291.32	
13.71	265.38	265.38	1.713	2024.26	277.17	277.17	
13.75	252.89	252.89	1.674	2024.24	264.17	264.17	
13.79	241.73	241.73	1.636	2024.21	252.13	252.13	
13.83	231.49	231.49	1.601	2024.19	241.31	241.31	
13.88	221.65	221.65	1.568	2024.18	231.03	231.03	
13.92	213.69	213.69	1.538	2024.16	222.03	222.03	
13.96	206.83	206.83	1.511	2024.14	214.20	214.20	
14.00	200.49	200.49	1.487	2024.13	207.19	207.19	
14.04	193.89	193.89	1.463	2024.11	200.65	200.65	
14.08	186.69	186.69	1.439	2024.10	194.03	194.03	
14.13	179.67	179.67	1.414	2024.09	187.10	187.10	
14.17	173.69	173.69	1.389	2024.07	180.44	180.44	
14.21	167.67	167.67	1.366	2024.06	174.37	174.37	
14.25	161.61	161.61	1.343	2024.05	168.34	168.34	
14.29	155.75	155.75	1.320	2024.03	162.35	162.35	
14.33	150.73	150.73	1.298	2024.02	156.74	156.74	
14.38	146.07	146.07	1.278	2024.01	152.15	152.15	
14.42	141.75	141.75	1.257	2024.00	147.62	147.62	

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
14.46	137.79	137.79	1.237	2023.99	143.30	143.30
14.50	134.18	134.18	1.219	2023.98	139.27	139.27
14.54	130.39	130.39	1.202	2023.97	135.43	135.43
14.58	126.45	126.45	1.184	2023.96	131.91	131.91
14.63	122.51	122.51	1.164	2023.95	128.19	128.19
14.67	118.60	118.60	1.145	2023.94	124.36	124.36
14.71	114.78	114.78	1.125	2023.93	120.51	120.51
14.75	111.18	111.18	1.106	2023.92	116.73	116.73
14.79	107.78	107.78	1.087	2023.92	113.18	113.18
14.83	104.60	104.60	1.068	2023.91	109.83	109.83
14.88	101.51	101.51	1.051	2023.90	106.58	106.58
14.92	98.91	98.91	1.034	2023.89	103.53	103.53
14.96	96.54	96.54	1.019	2023.88	100.74	100.74
15.00	94.24	94.24	1.005	2023.88	98.18	98.18
15.04	91.84	91.84	0.991	2023.87	95.71	95.71
15.08	89.44	89.44	0.978	2023.86	93.40	93.40
15.13	86.95	86.95	0.964	2023.86	91.03	91.03
15.17	84.56	84.56	0.950	2023.85	88.63	88.63
15.21	82.13	82.13	0.936	2023.84	86.22	86.22
15.25	79.84	79.84	0.922	2023.84	83.84	83.84
15.29	77.71	77.71	0.908	2023.83	81.53	81.53
15.33	75.65	75.65	0.896	2023.83	79.32	79.32
15.38	73.74	73.74	0.883	2023.82	77.24	77.24
15.42	71.96	71.96	0.871	2023.81	75.36	75.36
15.46	70.36	70.36	0.860	2023.81	73.56	73.56
15.50	68.85	68.85	0.849	2023.80	71.87	71.87
15.54	67.31	67.31	0.839	2023.80	70.25	70.25
15.58	65.68	65.68	0.829	2023.79	68.64	68.64
15.63	64.05	64.05	0.819	2023.79	67.02	67.02
15.67	62.46	62.46	0.808	2023.78	65.41	65.41
15.71	60.90	60.90	0.798	2023.78	63.81	63.81
15.75	59.37	59.37	0.788	2023.77	62.24	62.24
15.79	57.97	57.97	0.779	2023.77	60.73	60.73
15.83	56.56	56.56	0.769	2023.77	59.35	59.35
15.88	55.17	55.17	0.760	2023.76	57.97	57.97
15.92	54.05	54.05	0.750	2023.76	56.63	56.63
15.96	52.97	52.97	0.742	2023.75	55.39	55.39
16.00	51.96	51.96	0.734	2023.75	54.23	54.23
16.04	50.88	50.88	0.726	2023.74	53.11	53.11
16.08	49.80	49.80	0.718	2023.74	52.01	52.01
16.13	48.74	48.74	0.711	2023.74	50.92	50.92
16.17	47.69	47.69	0.703	2023.73	49.85	49.85
16.21	46.64	46.64	0.696	2023.73	48.78	48.78
16.25	45.63	45.63	0.688	2023.73	47.73	47.73
16.29	44.64	44.64	0.681	2023.72	46.70	46.70
16.33	43.66	43.66	0.674	2023.72	45.69	45.69
16.38	42.79	42.79	0.667	2023.72	44.80	44.80

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
16.42	41.96	41.96	0.660	2023.71	43.92	43.92
16.46	41.18	41.18	0.654	2023.71	43.07	43.07
16.50	40.43	40.43	0.647	2023.71	42.25	42.25
16.54	39.66	39.66	0.641	2023.70	41.45	41.45
16.58	38.88	38.88	0.635	2023.70	40.67	40.67
16.63	38.15	38.15	0.629	2023.70	39.89	39.89
16.67	37.40	37.40	0.623	2023.70	39.12	39.12
16.71	36.70	36.70	0.617	2023.69	38.37	38.37
16.75	35.99	35.99	0.611	2023.69	37.64	37.64
16.79	35.32	35.32	0.606	2023.69	36.92	36.92
16.83	34.70	34.70	0.600	2023.69	36.23	36.23
16.88	34.06	34.06	0.595	2023.68	35.56	35.56
16.92	33.48	33.48	0.590	2023.68	34.91	34.91
16.96	32.94	32.94	0.585	2023.68	34.30	34.30
17.00	32.41	32.41	0.581	2023.68	33.71	33.71
17.04	31.87	31.87	0.576	2023.67	33.14	33.14
17.08	31.29	31.29	0.572	2023.67	32.58	32.58
17.13	30.73	30.73	0.567	2023.67	32.04	32.04
17.17	30.20	30.20	0.563	2023.67	31.54	31.54
17.21	29.69	29.69	0.558	2023.67	31.03	31.03
17.25	0.00	0.00	0.511	2023.64	25.90	25.90

Total Routing Mass Balance Discrepancy is -0.03%

Modified Puls Routing

Inflow Hydrograph: F/SWMM-100.hyd
Storage/Elevation Curve: F/Pond1.ES
Discharge/Elevation Curve: F/Pond1.EO

100-year storm routing

Basin Bypass Capacity = 0.0 cfs
Starting Pool Elevation = 2023.40 feet
Time Interval = 4.166667E-02 hours

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
11.00	66.92	66.92	0.000	2023.40	0.00	0.00
11.04	68.34	68.34	0.225	2023.51	4.75	4.75
11.08	70.66	70.66	0.426	2023.60	17.47	17.47
11.13	73.12	73.12	0.584	2023.68	34.18	34.18
11.17	75.77	75.77	0.697	2023.73	49.03	49.03
11.21	78.29	78.29	0.775	2023.77	60.16	60.16
11.25	80.75	80.75	0.827	2023.79	68.39	68.39
11.29	82.75	82.75	0.863	2023.81	74.11	74.11
11.33	84.52	84.52	0.889	2023.82	78.22	78.22
11.38	85.87	85.87	0.908	2023.83	81.40	81.40
11.42	86.96	86.96	0.921	2023.84	83.68	83.68
11.46	87.79	87.79	0.931	2023.84	85.36	85.36
11.50	88.36	88.36	0.938	2423.85	86.60	86.60
11.54	91.84	91.84	0.947	2023.85	88.19	88.19
11.58	97.74	97.74	0.965	2023.86	91.20	91.20
11.63	103.94	103.94	0.991	2023.87	95.59	95.59
11.67	110.30	110.30	1.021	2023.88	101.11	101.11
11.71	116.70	116.70	1.053	2023.90	107.05	107.05
11.75	122.46	122.46	1.086	2023.92	113.06	113.06
11.79	127.12	127.12	1.116	2023.93	118.85	118.85
11.83	130.85	130.85	1.143	2023.94	123.94	123.94
11.88	133.63	133.63	1.164	2023.95	128.10	128.10
11.92	135.77	135.77	1.181	2023.96	131.41	131.41
11.96	137.66	137.66	1.195	2023.97	134.07	134.07
12.00	139.56	139.56	1.206	2023.97	136.43	136.43
12.04	237.15	237.15	1.334	2024.04	166.01	166.01
12.08	449.05	449.05	1.738	2024.27	285.40	285.40
12.13	735.74	735.74	2.387	2024.63	522.44	522.44
12.17	1123.04	1123.04	3.210	2025.05	858.51	858.51
12.21	1298.55	1298.55	3.841	2025.34	1196.93	1196.93
12.25	1447.39	1447.39	4.119	2025.46	1387.09	1387.09
12.29	1492.23	1492.23	4.243	2025.52	1480.86	1480.86
12.33	1508.98	1508.98	4.270	2025.54	1504.31	1504.31
12.38	1520.49	1520.49	4.285	2025.54	1516.69	1516.69
12.42	1531.07	1531.07	4.298	2025.55	1527.48	1527.48
12.46	1538.15	1538.15	4.308	2025.56	1535.95	1535.95

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)	
12.50	1544.30	1544.30	4.315	2025.56	1542.21	1542.21	Max. WSEL
12.54	1424.18	1424.18	4.234	2025.52	1473.55	1473.55	
12.58	1257.84	1257.84	4.032	2025.42	1325.51	1325.51	
12.63	1181.50	1181.50	3.863	2025.35	1211.85	1211.85	
12.67	1132.81	1132.81	3.774	2025.31	1154.36	1154.36	
12.71	1106.98	1106.98	3.717	2025.28	1118.63	1118.63	
12.75	1087.35	1087.35	3.681	2025.26	1096.65	1096.65	
12.79	1075.34	1075.34	3.655	2025.25	1081.19	1081.19	
12.83	1065.43	1065.43	3.636	2025.24	1070.27	1070.27	
12.88	1049.77	1049.77	3.615	2025.23	1057.47	1057.47	
12.92	1017.00	1017.00	3.573	2025.21	1033.43	1033.43	
12.96	999.39	999.39	3.529	2025.19	1008.75	1008.75	
13.00	987.80	987.80	3.502	2025.18	993.93	993.93	
13.04	963.61	963.61	3.470	2025.17	976.35	976.35	
13.08	927.78	927.78	3.413	2025.14	947.72	947.72	
13.13	890.00	890.00	3.340	2025.11	912.76	912.76	
13.17	849.70	849.70	3.255	2025.07	876.37	876.37	
13.21	806.11	806.11	3.156	2025.02	836.80	836.80	
13.25	689.08	689.08	2.979	2024.93	761.42	761.42	
13.29	589.32	589.32	2.734	2024.81	658.85	658.85	
13.33	537.91	537.91	2.540	2024.71	581.13	581.13	
13.38	507.20	507.20	2.419	2024.65	534.40	534.40	
13.42	481.74	481.74	2.335	2024.60	503.02	503.02	
13.46	459.97	459.97	2.268	2024.57	478.00	478.00	
13.50	441.16	441.16	2.209	2024.54	456.99	456.99	
13.54	421.65	421.65	2.155	2024.51	437.25	437.25	
13.58	400.23	400.23	2.100	2024.48	416.62	416.62	
13.63	378.09	378.09	2.043	2024.45	394.94	394.94	
13.67	357.60	357.60	1.987	2024.42	373.54	373.54	
13.71	339.32	339.32	1.934	2024.39	354.07	354.07	
13.75	322.83	322.83	1.885	2024.36	336.40	336.40	
13.79	308.57	308.57	1.841	2024.33	320.68	320.68	
13.83	295.73	295.73	1.801	2024.31	306.78	306.78	
13.88	284.86	284.86	1.765	2024.29	294.56	294.56	
13.92	275.50	275.50	1.734	2024.27	283.91	283.91	
13.96	266.81	266.81	1.706	2024.25	274.72	274.72	
14.00	258.80	258.80	1.680	2024.24	266.13	266.13	
14.04	250.34	250.34	1.654	2024.22	257.80	257.80	
14.08	241.15	241.15	1.627	2024.21	249.32	249.32	
14.13	231.53	231.53	1.598	2024.19	240.27	240.27	
14.17	221.56	221.56	1.567	2024.17	230.70	230.70	
14.21	213.16	213.16	1.537	2024.16	221.73	221.73	
14.25	205.67	205.67	1.509	2024.14	213.54	213.54	
14.29	198.85	198.85	1.483	2024.13	206.04	206.04	
14.33	192.62	192.62	1.459	2024.11	199.35	199.35	
14.38	186.96	186.96	1.436	2024.10	193.24	193.24	
14.42	181.94	181.94	1.416	2024.09	187.63	187.63	

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
14.46	177.99	177.99	1.398	2024.08	182.73	182.73
14.50	174.08	174.08	1.382	2024.07	178.49	178.49
14.54	169.65	169.65	1.366	2024.06	174.38	174.38
14.58	164.76	164.76	1.349	2024.05	169.93	169.93
14.63	159.72	159.72	1.331	2024.04	165.16	165.16
14.67	154.72	154.72	1.312	2024.03	160.24	160.24
14.71	150.10	150.10	1.293	2024.02	155.57	155.57
14.75	145.74	145.74	1.274	2024.01	151.36	151.36
14.79	141.55	141.55	1.255	2024.00	147.11	147.11
14.83	137.70	137.70	1.236	2023.99	142.99	142.99
14.88	134.26	134.26	1.218	2023.98	139.13	139.13
14.92	131.08	131.08	1.202	2023.97	135.58	135.58
14.96	128.07	128.07	1.187	2023.96	132.56	132.56
15.00	125.31	125.31	1.172	2023.96	129.61	129.61
15.04	122.43	122.43	1.157	2023.95	126.73	126.73
15.08	119.33	119.33	1.142	2023.94	123.80	123.80
15.13	116.23	116.23	1.126	2023.93	120.78	120.78
15.17	113.13	113.13	1.111	2023.93	117.72	117.72
15.21	110.13	110.13	1.095	2023.92	114.68	114.68
15.25	107.20	107.20	1.079	2023.91	111.80	111.80
15.29	104.53	104.53	1.064	2023.90	108.95	108.95
15.33	101.93	101.93	1.049	2023.90	106.21	106.21
15.38	99.71	99.71	1.034	2023.89	103.62	103.62
15.42	97.59	97.59	1.021	2023.89	101.24	101.24
15.46	95.66	95.66	1.009	2023.88	99.03	99.03
15.50	93.89	93.89	0.998	2023.87	96.99	96.99
15.54	91.91	91.91	0.987	2023.87	95.06	95.06
15.58	89.85	89.85	0.976	2023.86	93.16	93.16
15.63	87.78	87.78	0.965	2023.86	91.18	91.18
15.67	85.70	85.70	0.953	2023.85	89.16	89.16
15.71	83.62	83.62	0.941	2023.85	87.11	87.11
15.75	81.63	81.63	0.929	2023.84	85.07	85.07
15.79	79.77	79.77	0.917	2023.84	83.08	83.08
15.83	77.98	77.98	0.906	2023.83	81.17	81.17
15.88	76.36	76.36	0.896	2023.83	79.35	79.35
15.92	74.84	74.84	0.886	2023.82	77.64	77.64
15.96	73.43	73.43	0.876	2023.82	76.14	76.14
16.00	72.16	72.16	0.867	2023.81	74.71	74.71
16.04	70.78	70.78	0.858	2023.81	73.32	73.32
16.08	69.33	69.33	0.850	2023.80	71.92	71.92
16.13	67.89	67.89	0.841	2023.80	70.50	70.50
16.17	66.43	66.43	0.832	2023.79	69.07	69.07
16.21	65.01	65.01	0.822	2023.79	67.64	67.64
16.25	63.64	63.64	0.814	2023.79	66.22	66.22
16.29	62.34	62.34	0.805	2023.78	64.84	64.84
16.33	61.09	61.09	0.796	2023.78	63.50	63.50
16.38	59.92	59.92	0.788	2023.77	62.22	62.22

Event Time (hours)	Hydrograph Inflow (cfs)	Basin Inflow (cfs)	Storage Used (acre-ft)	Elevation Above MSL (feet)	Basin Outflow (cfs)	Outflow Total (cfs)
16.42	58.86	58.86	0.781	2023.77	61.01	61.01
16.46	57.84	57.84	0.773	2023.77	59.95	59.95
16.50	56.91	56.91	0.766	2023.76	58.92	58.92
16.54	55.85	55.85	0.759	2023.76	57.91	57.91
16.58	54.81	54.81	0.752	2023.76	56.89	56.89
16.63	53.76	53.76	0.745	2023.75	55.85	55.85
16.67	52.71	52.71	0.738	2023.75	54.81	54.81
16.71	51.71	51.71	0.730	2023.75	53.78	53.78
16.75	50.78	50.78	0.723	2023.74	52.77	52.77
16.79	49.87	49.87	0.717	2023.74	51.80	51.80
16.83	48.98	48.98	0.710	2923.74	50.86	50.86
16.88	48.11	48.11	0.704	2023.73	49.94	49.94
16.92	47.30	47.30	0.698	2023.73	49.05	49.05
16.96	46.60	46.60	0.692	2023.73	48.22	48.22
17.00	45.92	45.92	0.686	2023.73	47.44	47.44
17.04	45.16	45.16	0.681	2023.72	46.68	46.68
17.08	44.38	44.38	0.676	2023.72	45.92	45.92
17.13	43.60	43.60	0.670	2023.72	45.21	45.21
17.17	42.84	42.84	0.665	2023.72	44.49	44.49
17.21	42.08	42.08	0.659	2023.71	43.76	43.76
17.25	0.00	0.00	0.595	2023.68	35.54	35.54

Total Routing Mass Balance Discrepancy is -0.03%

Swale Design

Input Information

Normal Depth = 1.430 feet
Manning's n = 0.035
Channel Grade = 0.05 ft/ft
Left Sideslope = 6H:1V
Right Sideslope = 6H:1V
Bottom Width = 50 feet
Channel Capacity = 922 cfs ← 2-year storm peak outflow

Solution Output

Area = 83.78 square feet
Hydraulic Radius = 1.24 feet
Froude Number = 1 .74
Velocity = 11.005 ft/s
VxRh = 13.68 square feet/s
Top Width = 67.16 feet
Critical Depth = 2.02 feet
Rip Rap Size (D_{50}) = 36.54 inches

C.2 LOWER POND MODEL

Following are hydrographs calculated for the lower pond using the SCS Unit Hydrograph method in VTPSUHM and the parameters listed in section 2.6.

SCS Unit Hydrograph

Title: Lower pond
Hyetograph: F:\2YR.HYT | **Return Period: F:\2YR.HYT**
Drainage Area: 2.95 sq. mi. | **Curve Number: 82**
Time of Concentration: 1.25 hrs | **Storm Length: 24 hrs**
Storm Start: 01/07/2003, hr 0000 | **UH Factor: 484**

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)	
1/7/2003	0940	0.00	0.0000	0.00	
1/7/2003	0950	171.20	0.0005	0.09	
1/7/2003	1000	530.71	0.0006	0.38	
1/7/2003	1010	1129.91	0.0009	1.06	
1/7/2003	1020	1592.14	0.0012	2.19	
1/7/2003	1030	1711.98	0.0015	3.77	
1/7/2003	1040	1592.14	0.0022	5.84	
1/7/2003	1050	1335.35	0.0026	8.43	
1/7/2003	1100	958.71	0.0030	11.61	
1/7/2003	1110	667.67	0.0047	15.55	
1/7/2003	1120	479.35	0.0054	20.37	
1/7/2003	1130	354.35	0.0060	26.34	
1/7/2003	1140	251.66	0.0105	33.98	
1/7/2003	1150	183.18	0.0120	43.73	
1/7/2003	1200	131.82	0.0133	56.46	
1/7/2003	1210	94.16	0.1445	94.12	
1/7/2003	1220	68.48	0.1977	188.83	
1/7/2003	1230	49.65	0.2347	386.70	
1/7/2003	1240	35.95	0.0477	656.30	
1/7/2003	1250	27.39	0.0486	918.43	
1/7/2003	1300	20.54	0.0495	1074.06	
1/7/2003	1310	17.12	0.0257	$\boxed{1102.31}$	Max. discharge
1/7/2003	1320	11.98	0.0259	1031.59	
1/7/2003	1330	5.14	0.0261	900.86	
1/7/2003	1340	1.71	0.0192	751.30	
1/7/2003	1350	0.00	0.0193	628.98	
1/7/2003	1400		0.0194	531.88	
1/7/2003	1410		0.0152	452.31	
1/7/2003	1420		0.0152	387.92	
1/7/2003	1430		0.0153	337.27	
1/7/2003	1440		0.0109	295.31	
1/7/2003	1450		0.0110	260.95	
1/7/2003	1500		0.0110	232.02	
1/7/2003	1510		0.0110	207.15	
1/7/2003	1520		0.0111	186.96	
1/7/2003	1530		0.0111	171.44	

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	1540		0.0111	159.32
1/7/2003	1550		0.0111	150.10
1/7/2003	1600		0.0112	142.77
1/7/2003	1610		0.0112	137.20
1/7/2003	1620		0.0112	133.74
1/7/2003	1630		0.0112	131.61
1/7/2003	1640		0.0068	129.60
1/7/2003	1650		0.0068	126.48
1/7/2003	1700		0.0068	120.95
1/7/2003	1710		0.0068	113.55
1/7/2003	1720		0.0068	105.73
1/7/2003	1730		0.0068	98.52
1/7/2003	1740		0.0068	92.50
1/7/2003	1750		0.0068	88.23
1/7/2003	1800		0.0068	85.28
1/7/2003	1810		0.0068	83.20
1/7/2003	1820		0.0068	81.70
1/7/2003	1830		0.0069	80.67
1/7/2003	1840		0.0069	79.96
1/7/2003	1850		0.0069	79.47
1/7/2003	1900		0.0069	79.15
1/7/2003	1910		0.0069	78.94
1/7/2003	1920		0.0069	78.82
1/7/2003	1930		0.0069	78.76
1/7/2003	1940		0.0069	78.73
1/7/2003	1950		0.0069	78.74
1/7/2003	2000		0.0069	78.76
1/7/2003	2010		0.0069	78.80
1/7/2003	2020		0.0069	78.87
1/7/2003	2030		0.0070	78.96
1/7/2003	2040		0.0046	78.66
1/7/2003	2050		0.0046	77.52
1/7/2003	2100		0.0046	74.98
1/7/2003	2110		0.0047	71.37
1/7/2003	2120		0.0047	67.47
1/7/2003	2130		0.0047	63.84
1/7/2003	2140		0.0047	60.80
1/7/2003	2150		0.0047	58.63
1/7/2003	2200		0.0047	57.13
1/7/2003	2210		0.0047	56.06
1/7/2003	2220		0.0047	55.28
1/7/2003	2230		0.0047	54.74
1/7/2003	2240		0.0047	54.36
1/7/2003	2250		0.0047	54.09
1/7/2003	2300		0.0047	53.92
1/7/2003	2310		0.0047	53.80
1/7/2003	2320		0.0047	53.72

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	2330		0.0047	53.68
1/7/2003	2340		0.0047	53.65
1/7/2003	2350		0.0047	53.64
1/8/2003	0000		0.0047	53.64
1/8/2003	0010			52.85
1/8/2003	0020			50.37
1/8/2003	0030			45.07
1/8/2003	0040			37.60
1/8/2003	0050			29.56
1/8/2003	0100			22.07
1/8/2003	0110			15.79
1/8/2003	0120			11.29
1/8/2003	0130			8.15
1/8/2003	0140			5.89
1/8/2003	0150			4.23
1/8/2003	0200			3.04
1/8/2003	0210			2.18
1/8/2003	0220			1.56
1/8/2003	0230			1.12
1/8/2003	0240			0.80
1/8/2003	0250			0.56
1/8/2003	0300			0.39
1/8/2003	0310			0.27
1/8/2003	0320			0.17
1/8/2003	0330			0.09
1/8/2003	0340			0.03
1/8/2003	0350			0.01
1/8/2003	0400			0.00

SCS Unit Hydrograph

Title: Lower pond
Hyetograph: F:\10YR.HYT | **Return Period: F:\10YR.HYT**
Drainage Area: 2.95 sq. mi. | **Curve Number: 82**
Time of Concentration: 1.25 hrs | **Storm Length: 24 hrs**
Storm Start: 01/07/2003, hr 0000 | **UH Factor: 484**

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)	
1/7/2003	720	0.00	0.0000	0.00	
1/7/2003	730	171.20	0.0010	0.18	
1/7/2003	740	530.71	0.0009	0.70	
1/7/2003	750	1129.91	0.0011	1.84	
1/7/2003	800	1592.14	0.0014	3.48	
1/7/2003	810	1711.98	0.0018	5.47	
1/7/2003	820	1592.14	0.0021	7.77	
1/7/2003	830	1335.35	0.0023	10.35	
1/7/2003	840	958.71	0.0029	13.13	
1/7/2003	850	667.67	0.0032	16.17	
1/7/2003	900	479.35	0.0035	19.50	
1/7/2003	910	354.35	0.0042	23.14	
1/7/2003	920	251.66	0.0046	27.05	
1/7/2003	930	183.18	0.0050	31.29	
1/7/2003	940	131.82	0.0061	35.93	
1/7/2003	950	94.16	0.0065	40.95	
1/7/2003	1000	68.48	0.0069	46.45	
1/7/2003	1010	49.65	0.0086	52.54	
1/7/2003	1020	35.95	0.0092	59.18	
1/7/2003	1030	27.39	0.0097	66.56	
1/7/2003	1040	20.54	0.0125	74.87	
1/7/2003	1050	17.12	0.0132	84.13	
1/7/2003	1100	11.98	0.0139	94.69	
1/7/2003	1110	5.14	0.0194	107.06	
1/7/2003	1120	1.71	0.0204	121.32	
1/7/2003	1130	0.00	0.0215	138.30	
1/7/2003	1140		0.0355	159.57	
1/7/2003	1150		0.0377	186.04	
1/7/2003	1200		0.0397	220.26	
1/7/2003	1210		0.3702	316.87	
1/7/2003	1220		0.4408	544.91	
1/7/2003	1230		0.4852	1000.54	
1/7/2003	1240		0.0954	1594.63	
1/7/2003	1250		0.0964	2139.95	
1/7/2003	1300		0.0973	2435.02	
1/7/2003	1310		0.0508	2450.02	Max. discharge

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	1320		0.0510	2251.34
1/7/2003	1330		0.0512	1935.46
1/7/2003	1340		0.0366	1594.63
1/7/2003	1350		0.0367	1320.85
1/7/2003	1400		0.0368	1104.84
1/7/2003	1410		0.0292	929.51
1/7/2003	1420		0.0293	788.67
1/7/2003	1430		0.0294	678.82
1/7/2003	1440		0.0246	589.89
1/7/2003	1450		0.0246	519.52
1/7/2003	1500		0.0246	463.60
1/7/2003	1510		0.0214	417.88
1/7/2003	1520		0.0214	381.11
1/7/2003	1530		0.0214	351.58
1/7/2003	1540		0.0190	325.95
1/7/2003	1550		0.0190	303.26
1/7/2003	1600		0.0191	282.30
1/7/2003	1610		0.0172	263.79
1/7/2003	1620		0.0172	249.50
1/7/2003	1630		0.0172	237.69
1/7/2003	1640		0.0158	227.54
1/7/2003	1650		0.0158	218.75
1/7/2003	1700		0.0158	210.85
1/7/2003	1710		0.0145	203.55
1/7/2003	1720		0.0145	197.01
1/7/2003	1730		0.0146	190.91
1/7/2003	1740		0.0135	185.10
1/7/2003	1750		0.0136	179.80
1/7/2003	1800		0.0136	174.84
1/7/2003	1810		0.0127	170.11
1/7/2003	1820		0.0127	165.81
1/7/2003	1830		0.0127	161.76
1/7/2003	1840		0.0120	157.87
1/7/2003	1850		0.0120	154.31
1/7/2003	1900		0.0120	150.93
1/7/2003	1910		0.0114	147.70
1/7/2003	1920		0.0114	144.73
1/7/2003	1930		0.0114	141.95
1/7/2003	1940		0.0109	139.27
1/7/2003	1950		0.0109	136.79
1/7/2003	2000		0.0109	134.39
1/7/2003	2010		0.0103	132.02
1/7/2003	2020		0.0103	129.75
1/7/2003	2030		0.0103	127.51
1/7/2003	2040		0.0099	125.26
1/7/2003	2050		0.0099	123.15
1/7/2003	2100		0.0099	121.12

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	2110		0.0094	119.14
1/7/2003	2120		0.0094	117.30
1/7/2003	2130		0.0094	115.51
1/7/2003	2140		0.0090	113.74
1/7/2003	2150		0.0090	112.04
1/7/2003	2200		0.0090	110.35
1/7/2003	2210		0.0087	108.67
1/7/2003	2220		0.0087	107.10
1/7/2003	2230		0.0087	105.61
1/7/2003	2240		0.0084	104.19
1/7/2003	2250		0.0084	102.88
1/7/2003	2300		0.0084	101.64
1/7/2003	2310		0.0081	100.41
1/7/2003	2320		0.0081	99.25
1/7/2003	2330		0.0082	98.11
1/7/2003	2340		0.0079	96.95
1/7/2003	2350		0.0079	95.84
1/8/2003	0000		0.0079	94.73
1/8/2003	0010			92.31
1/8/2003	0020			87.20
1/8/2003	0030			77.54
1/8/2003	0040			64.42
1/8/2003	0050			50.52
1/8/2003	0100			37.69
1/8/2003	0110			26.97
1/8/2003	0120			19.27
1/8/2003	0130			13.90
1/8/2003	0140			10.05
1/8/2003	0150			7.21
1/8/2003	0200			5.19
1/8/2003	0210			3.71
1/8/2003	0220			2.66
1/8/2003	0230			1.90
1/8/2003	0240			1.35
1/8/2003	0250			0.95
1/8/2003	0300			0.66
1/8/2003	0310			0.45
1/8/2003	0320			0.28
1/8/2003	0330			0.15
1/8/2003	0340			0.05
1/8/2003	0350			0.01
1/8/2003	0400			0.00

SCS Unit Hydrograph

Title: Lower pond
Hyetograph: F:\25YR.HYT | **Return Period: F:\25YR.HYT**
Drainage Area: 2.95 sq. mi. | **Curve Number: 82**
Time of Concentration: 1.25 hrs | **Storm Length: 24 hrs**
Storm Start: 01/07/2003, hr 0000 | **UH Factor: 484**

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	0650	0.00	0.0000	0.00
1/7/2003	0700	171.2	0.0009	0.15
1/7/2003	0710	530.71	0.0008	0.62
1/7/2003	0720	1129.91	0.0010	1.65
1/7/2003	0730	1592.14	0.0014	3.19
1/7/2003	0740	1711.98	0.0017	5.13
1/7/2003	0750	1592.14	0.0020	7.42
1/7/2003	0800	1335.35	0.0023	10.02
1/7/2003	0810	958.71	0.0028	12.84
1/7/2003	0830	667.67	0.0031	15.90
1/7/2003	0830	479.35	0.0034	19.24
1/7/2003	0840	354.35	0.0041	22.86
1/7/2003	0850	251.66	0.0045	26.72
1/7/2003	0900	183.18	0.0048	30.88
1/7/2003	0910	131.82	0.0058	35.36
1/7/2003	0920	94.16	0.0062	40.13
1/7/2003	0930	68.48	0.0066	45.28
1/7/2003	0940	49.65	0.0080	50.88
1/7/2003	0950	35.95	0.0085	56.90
1/7/2003	1000	27.39	0.0089	63.48
1/7/2003	1010	20.54	0.0110	70.76
1/7/2003	1020	17.12	0.0116	78.67
1/7/2003	1030	11.98	0.0122	87.46
1/7/2003	1040	5.14	0.0156	97.37
1/7/2003	1050	1.71	0.0164	108.37
1/7/2003	1100	0.00	0.0172	120.90
1/7/2003	1110		0.0238	135.56
1/7/2003	1120		0.0250	152.44
1/7/2003	1130		0.0261	172.53
1/7/2003	1140		0.0429	197.71
1/7/2003	1150		0.0453	228.99
1/7/2003	1200		0.0474	269.40
1/7/2003	1210		0.4336	382.65
1/7/2003	1220		0.5069	647.70
1/7/2003	1230		0.5521	1174.03
1/7/2003	1240		0.1081	1856.11

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)	
1/7/2003	1250		0.1091	2476.98	
1/7/2003	1300		0.1100	2808.03	
1/7/2003	1310		0.0575	2817.65	Max. discharge
1/7/2003	1320		0.0577	2582.67	
1/7/2003	1330		0.0579	2215.54	
1/7/2003	1340		0.0412	1822.40	
1/7/2003	1350		0.0413	1507.35	
1/7/2003	1400		0.0414	1258.97	
1/7/2003	1410		0.0329	1057.60	
1/7/2003	1420		0.0330	896.00	
1/7/2003	1430		0.0331	770.06	
1/7/2003	1440		0.0277	668.30	
1/7/2003	1450		0.0278	587.94	
1/7/2003	1500		0.0278	524.22	
1/7/2003	1510		0.0240	472.25	
1/7/2003	1520		0.0241	430.46	
1/7/2003	1530		0.0241	396.89	
1/7/2003	1540		0.0215	367.71	
1/7/2003	1550		0.0215	341.85	
1/7/2003	1600		0.0215	318.05	
1/7/2003	1610		0.0193	297.12	
1/7/2003	1620		0.0193	281.00	
1/7/2003	1630		0.0193	267.67	
1/7/2003	1640		0.0177	256.16	
1/7/2003	1650		0.0178	246.15	
1/7/2003	1700		0.0178	237.13	
1/7/2003	1710		0.0164	228.82	
1/7/2003	1720		0.0165	221.45	
1/7/2003	1730		0.0165	214.68	
1/7/2003	1740		0.0153	208.31	
1/7/2003	1750		0.0153	202.53	
1/7/2003	1800		0.0153	197.09	
1/7/2003	1810		0.0143	191.85	
1/7/2003	1820		0.0143	187.02	
1/7/2003	1830		0.0143	182.40	
1/7/2003	1840		0.0134	177.92	
1/7/2003	1850		0.0134	173.77	
1/7/2003	1900		0.0134	169.80	
1/7/2003	1910		0.0127	165.94	
1/7/2003	1920		0.0127	162.38	
1/7/2003	1930		0.9127	158.98	
1/7/2003	1940		0.0121	155.71	
1/7/2003	1950		0.0121	152.70	
1/7/2003	2000		0.0121	149.87	
1/7/2003	2010		0.0115	147.14	
1/7/2003	2020		0.0115	144.60	
1/7/2003	2030		0.0115	142.16	

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	2040		0.0111	139.75
1/7/2003	2050		0.0111	137.52
1/7/2003	2100		0.0111	135.39
1/7/2003	2110		0.0106	133.33
1/7/2003	2120		0.0107	131.42
1/7/2003	2130		0.0107	129.58
1/7/2003	2140		0.0102	127.75
1/7/2003	2150		0.0102	126.01
1/7/2003	2200		0.0102	124.29
1/7/2003	2210		0.0098	122.54
1/7/2003	2220		0.0098	120.86
1/7/2003	2230		0.0098	119.17
1/7/2003	2240		0.0095	117.48
1/7/2003	2250		0.0095	115.89
1/7/2003	2300		0.0095	114.39
1/7/2003	2310		0.0092	112.94
1/7/2003	2320		0.0092	111.61
1/7/2003	2330		0.0092	110.35
1/7/2003	2340		0.0089	109.10
1/7/2003	2350		0.0089	107.92
1/8/2003	0000		0.0089	106.75
1/8/2003	0010			104.10
1/8/2003	0020			98.40
1/8/2003	0030			87.55
1/8/2003	0040			72.75
1/8/2003	0050			57.07
1/8/2003	0100			42.58
1/8/2003	0110			30.47
1/8/2003	0120			21.77
1/8/2003	0130			15.71
1/8/2003	0140			11.36
1/8/2003	0150			8.14
1/8/2003	0200			5.86
1/8/2003	0210			4.20
1/8/2003	0220			3.00
1/8/2003	0230			2.15
1/8/2003	0240			1.53
1/8/2003	0250			1.08
1/8/2003	0300			0.75
1/8/2003	0310			0.50
1/8/2003	0320			0.32
1/8/2003	0330			0.17
1/8/2003	0340			0.06
1/8/2003	0350			0.02
1/8/2003	0400			0.00

SCS Unit Hydrograph

Title: Lower pond
Hyetograph: F:\50YR.HYT | **Return Period: F:\50YR.HYT**
Drainage Area: 2.95 sq. mi. | **Curve Number: 82**
Time of Concentration: 1.25 hrs | **Storm Length: 24 hrs**
Storm Start: 01/07/2003, hr 0000 | **UH Factor: 484**

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	0620	0.00	0.0000	0.00
1/7/2003	0630	171.2	0.0009	0.15
1/7/2003	0640	530.71	0.0009	0.60
1/7/2003	0650	1129.91	0.0011	1.63
1/7/2003	0700	1592.14	0.0014	3.19
1/7/2003	0710	1711.98	0.0018	5.19
1/7/2003	0720	1592.14	0.0021	7.57
1/7/2003	0730	1335.35	0.0024	10.30
1/7/2003	0740	958.71	0.0029	13.26
1/7/2003	0750	667.67	0.0033	16.47
1/7/2003	0800	479.35	0.0036	19.96
1/7/2003	0810	354.35	0.0042	23.73
1/7/2003	0820	251.66	0.0046	27.71
1/7/2003	0830	183.18	0.0049	31.95
1/7/2003	0840	131.82	0.0058	36.46
1/7/2003	0850	94.16	0.0062	41.22
1/7/2003	0900	68.48	0.0066	46.29
1/7/2003	0910	49.65	0.0079	51.75
1/7/2003	0920	35.95	0.0083	57.55
1/7/2003	0930	27.39	0.0088	63.83
1/7/2003	0940	20.54	0.0105	70.66
1/7/2003	0950	17.12	0.0110	77.96
1/7/2003	1000	11.98	0.0115	85.90
1/7/2003	1010	5.14	0.0141	94.62
1/7/2003	1020	1.71	0.0148	104.06
1/7/2003	1030	0.00	0.0154	114.56
1/7/2003	1040		0.0196	126.41
1/7/2003	1050		0.0205	139.56
1/7/2003	1100		0.0213	154.55
1/7/2003	1110		0.0293	172.06
1/7/2003	1120		0.0306	192.17
1/7/2003	1130		0.0318	216.08
1/7/2003	1140		0.0519	246.00
1/7/2003	1150		0.0545	283.06
1/7/2003	1200		0.0568	330.82
1/7/2003	1210		0.5111	464.13

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)	
1/7/2003	1220		0.5867	773.92	
1/7/2003	1230		0.6325	1385.83	
1/7/2003	1240		0.1234	2174.17	
1/7/2003	1250		0.1244	2885.89	
1/7/2003	1300		0.1253	3259.90	
1/7/2003	1310		0.0652	3262.51	Max. discharge
1/7/2003	1320		0.0654	2983.08	
1/7/2003	1330		0.0657	2553.47	
1/7/2003	1340		0.0468	2096.66	
1/7/2003	1350		0.0469	1731.43	
1/7/2003	1400		0.0470	1443.85	
1/7/2003	1410		0.0373	1211.19	
1/7/2003	1420		0.0374	1024.75	
1/7/2003	1430		0.0375	879.61	
1/7/2003	1440		0.0314	762.46	
1/7/2003	1450		0.0314	670.02	
1/7/2003	1500		0.0315	596.76	
1/7/2003	1510		0.0273	537.12	
1/7/2003	1520		0.0274	489.27	
1/7/2003	1530		0.0274	450.99	
1/7/2003	1540		0.0243	417.78	
1/7/2003	1550		0.0243	388.29	
1/7/2003	1600		0.0243	361.09	
1/7/2003	1610		0.0219	337.13	
1/7/2003	1620		0.0219	318.69	
1/7/2003	1630		0.0220	303.48	
1/7/2003	1640		0.0200	290.40	
1/7/2003	1650		0.0200	279.03	
1/7/2003	1700		0.0200	268.74	
1/7/2003	1710		0.0186	259.19	
1/7/2003	1720		0.0186	250.68	
1/7/2003	1730		0.0186	242.84	
1/7/2003	1740		0.0172	235.47	
1/7/2003	1750		0.0172	228.80	
1/7/2003	1800		0.0172	222.54	
1/7/2003	1810		0.0162	216.54	
1/7/2003	1820		0.0162	211.05	
1/7/2003	1830		0.0162	205.89	
1/7/2003	1840		0.0152	200.94	
1/7/2003	1850		0.0152	196.38	
1/7/2003	1900		0.0152	191.98	
1/7/2003	1910		0.0144	187.68	
1/7/2003	1920		0.0144	183.69	
1/7/2003	1930		0.0144	179.93	
1/7/2003	1940		0.0137	176.32	
1/7/2003	1950		0.0137	173.00	
1/7/2003	2000		0.0137	169.81	

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	2010		0.0131	166.70
1/7/2003	2020		0.0131	163.80
1/7/2003	2030		0.0131	161.01
1/7/2003	2040		0.0125	158.31
1/7/2003	2050		0.0125	155.78
1/7/2003	2100		0.0125	153.32
1/7/2003	2110		0.0119	150.87
1/7/2003	2120		0.0119	148.52
1/7/2003	2130		0.0119	146.19
1/7/2003	2140		0.0114	143.85
1/7/2003	2150		0.0114	141.65
1/7/2003	2200		0.0115	139.53
1/7/2003	2210		0.0110	137.47
1/7/2003	2220		0.0110	135.55
1/7/2003	2230		0.0110	133.68
1/7/2003	2240		0.0107	131.86
1/7/2003	2250		0.0197	130.17
1/7/2003	2300		0.0107	128.58
1/7/2003	2310		0.0103	127.04
1/7/2003	2320		0.0103	125.58
1/7/2003	2330		0.0103	124.10
1/7/2003	2340		0.0100	122.57
1/7/2003	2350		0.0100	121.10
1/8/2003	0000		0.0100	119.66
1/8/2003	0010			116.60
1/8/2003	0020			110.17
1/8/2003	0030			98.99
1/8/2003	0040			81.43
1/8/2003	0050			63.87
1/8/2003	0100			47.66
1/8/2003	0110			34.10
1/8/2003	0120			24.36
1/8/2003	0130			17.58
1/8/2003	0140			12.71
1/8/2003	0150			9.11
1/8/2003	0200			6.56
1/8/2003	0210			4.70
1/8/2003	0220			3.36
1/8/2003	0230			2.40
1/8/2003	0240			1.71
1/8/2003	0250			1.20
1/8/2003	0300			0.84
1/8/2003	0310			0.56
1/8/2003	0320			0.36
1/8/2003	0330			0.19
1/8/2003	0340			0.07
1/8/2003	0350			0.02

SCS Unit Hydrograph

Title: Lower pond
Hyetograph: F:\100YR.HYT | **Return Period: F:\100YR.HYT**
Drainage Area: 2.95 sq. mi. | **Curve Number: 82**
Time of Concentration: 1.25 hrs | **Storm Length: 24 hrs**
Storm Start: 01/07/2003, hr 0000 | **UH Factor: 484**

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	0600	0.00	0.0000	0.00
1/7/2003	0610	171.20	0.0010	0.17
1/7/2003	0620	530.71	0.0009	0.67
1/7/2003	0630	1129.91	0.0012	1.79
1/7/2003	0640	1592.14	0.0016	3.50
1/7/2003	0650	1711.98	0.0019	5.67
1/7/2003	0700	1592.14	0.0022	8.26
1/7/2003	0710	1335.35	0.0027	11.21
1/7/2003	0720	958.71	0.0030	14.38
1/7/2003	0730	667.67	0.0034	17.82
1/7/2003	0740	479.35	0.0040	21.51
1/7/2003	0750	354.35	0.0043	25.44
1/7/2003	0800	251.66	0.0047	29.58
1/7/2003	0810	183.18	0.0055	33.99
1/7/2003	0820	131.82	0.0059	38.63
1/7/2003	0830	94.16	0.0063	43.56
1/7/2003	0840	68.48	0.0073	48.82
1/7/2003	0850	49.65	0.0078	54.35
1/7/2003	0900	35.95	0.0082	60.24
1/7/2003	0910	27.39	0.0097	66.55
1/7/2003	0920	20.54	0.0102	73.22
1/7/2003	0930	17.12	0.0106	80.41
1/7/2003	0940	11.98	0.0127	88.22
1/7/2003	0950	5.14	0.0132	96.57
1/7/2003	1000	1.71	0.0138	105.67
1/7/2003	1010	0.00	0.0168	115.70
1/7/2003	1020		0.0175	126.54
1/7/2003	1030		0.0182	138.54
1/7/2003	1040		0.0232	152.07
1/7/2003	1050		0.0241	167.04
1/7/2003	1100		0.0249	184.11
1/7/2003	1110		0.0341	204.07
1/7/2003	1120		0.0355	226.97
1/7/2003	1130		0.0368	254.15
1/7/2003	1140		0.0598	288.11
1/7/2003	1150		0.0625	330.11

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)	
1/7/2003	1200		0.0649	384.22	
1/7/2003	1210		0.5765	534.62	
1/7/2003	1220		0.6536	882.21	
1/7/2003	1230		0.6995	1566.00	
1/7/2003	1240		0.1359	2443.19	
1/7/2003	1250		0.1369	3230.38	
1/7/2003	1300		0.1378	3639.44	Max. discharge
1/7/2003	1310		0.0719	3635.27	
1/7/2003	1320		0.0721	3317.91	
1/7/2003	1330		0.0723	2835.73	
1/7/2003	1340		0.0514	2325.75	
1/7/2003	1350		0.0515	1918.73	
1/7/2003	1400		0.0516	1598.45	
1/7/2003	1410		0.0410	1339.56	
1/7/2003	1420		0.0411	1132.20	
1/7/2003	1430		0.0411	970.86	
1/7/2003	1440		0.0344	840.80	
1/7/2003	1450		0.0344	738.23	
1/7/2003	1500		0.0345	656.98	
1/7/2003	1510		0.0300	590.86	
1/7/2003	1520		0.0300	537.81	
1/7/2003	1530		0.0300	495.41	
1/7/2003	1540		0.0266	458.66	
1/7/2003	1550		0.0266	426.02	
1/7/2003	1600		0.0266	395.94	
1/7/2003	1610		0.0240	369.48	
1/7/2003	1620		0.0240	349.14	
1/7/2003	1630		0.0241	332.40	
1/7/2003	1640		0.0219	318.05	
1/7/2003	1650		0.0220	305.61	
1/7/2003	1700		0.0220	294.35	
1/7/2003	1710		0.0203	283.92	
1/7/2003	1720		0.0203	274.61	
1/7/2003	1730		0.0203	266.00	
1/7/2003	1740		0.0190	257.92	
1/7/2003	1750		0.0190	250.63	
1/7/2003	1800		0.0190	243.85	
1/7/2003	1810		0.0178	237.40	
1/7/2003	1820		0.0178	231.52	
1/7/2003	1830		0.0178	225.95	
1/7/2003	1840		0.0167	220.57	
1/7/2003	1850		0.0167	215.60	
1/7/2003	1900		0.0167	210.84	
1/7/2003	1910		0.0158	206.21	
1/7/2003	1920		0.0158	201.91	
1/7/2003	1930		0.0158	197.80	
1/7/2003	1940		0.0151	193.81	

Date	Time	Unit Hydrograph (cfs/in)	Rainfall Excess (in)	Flow (cfs)
1/7/2003	1950		0.0151	190.12
1/7/2003	2000		0.0151	186.61
1/7/2003	2010		0.0143	183.19
1/7/2003	2020		0.0143	180.00
1/7/2003	2030		0.0143	176.89
1/7/2003	2040		0.0137	173.82
1/7/2003	2050		0.0137	170.93
1/7/2003	2100		0.0137	168.16
1/7/2003	2110		0.0131	165.44
1/7/2003	2120		0.0131	162.90
1/7/2003	2130		0.0131	160.42
1/7/2003	2140		0.0125	157.94
1/7/2003	2150		0.0125	155.57
1/7/2003	2200		0.0125	153.21
1/7/2003	2210		0.0120	150.84
1/7/2003	2220		0.0120	148.61
1/7/2003	2230		0.0120	146.46
1/7/2003	2240		0.0117	144.39
1/7/2003	2250		0.0117	142.53
1/7/2003	2300		0.0117	140.81
1/7/2003	2310		0.0113	139.18
1/7/2003	2320		0.0113	137.64
1/7/2003	2330		0.0113	136.10
1/7/2003	2340		0.0108	134.50
1/7/2003	2350		0.0108	132.91
1/8/2003	0000		0.0108	131.26
1/8/2003	0010			127.78
1/8/2003	0020			120.60
1/8/2003	0030			107.18
1/8/2003	0040			89.00
1/8/2003	0050			69.78
1/8/2003	0100			52.06
1/8/2003	0110			37.25
1/8/2003	0120			26.61
1/8/2003	0130			19.20
1/8/2003	0140			13.88
1/8/2003	0150			9.95
1/8/2003	0200			7.16
1/8/2003	0210			5.13
1/8/2003	0220			3.67
1/8/2003	0230			2.62
1/8/2003	0240			1.86
1/8/2003	0250			1.32
1/8/2003	0300			0.92
1/8/2003	0310			0.62
1/8/2003	0320			0.39
1/8/2003	0330			0.20

Outlet Structure Configuration

Outlet Structure Configuration for Lower pond spillway

Stage 1: Rectangular Weir
Crest Elevation = 2019 feet
Length = 45 feet
Discharge Coefficient = 3.1
} Concrete spillway

Stage 2: Rectangular Weir
Crest Elevation = 2021.3 feet
Length = 225 feet
Discharge Coefficient = 3.1
} Grass above either side of concrete spillway

Basin Rating Curve

Basin Water Elevation	Basin Outflow (cfs)	Riser Box Water Elevation	Tailwater Elevation (ft)	Outfall Culvert Control	Outfall Culvert Override?
2019.00	0.00	N/A	N/A	N/A	N/A
2019.10	4.61	N/A	N/A	N/A	N/A
2019.20	12.76	N/A	N/A	N/A	N/A
2019.30	23.26	N/A	N/A	N/A	N/A
2019.40	35.68	N/A	N/A	N/A	N/A
2019.50	49.76	N/A	N/A	N/A	N/A
2019.60	65.31	N/A	N/A	N/A	N/A
2019.70	82.21	N/A	N/A	N/A	N/A
2019.80	100.36	N/A	N/A	N/A	N/A
2019.90	119.68	N/A	N/A	N/A	N/A
2020.00	140.10	N/A	N/A	N/A	N/A
2020.10	161.57	N/A	N/A	N/A	N/A
2020.20	184.03	N/A	N/A	N/A	N/A
2020.30	207.45	N/A	N/A	N/A	N/A
2020.40	231.78	N/A	N/A	N/A	N/A
2020.50	257.00	N/A	N/A	N/A	N/A
2020.60	283.07	N/A	N/A	N/A	N/A
2020.70	309.97	N/A	N/A	N/A	N/A
2020.80	337.67	N/A	N/A	N/A	N/A
2020.90	366.14	N/A	N/A	N/A	N/A
2021.00	395.38	N/A	N/A	N/A	N/A
2021.10	425.35	N/A	N/A	N/A	N/A
2021.20	456.05	N/A	N/A	N/A	N/A
2021.30	487.45	N/A	N/A	N/A	N/A
2021.40	542.41	N/A	N/A	N/A	N/A
2021.50	615.82	N/A	N/A	N/A	N/A
2021.60	701.71	N/A	N/A	N/A	N/A
2021.70	797.83	N/A	N/A	N/A	N/A
2021.80	902.85	N/A	N/A	N/A	N/A
2021.90	1015.90	N/A	N/A	N/A	N/A
2022.00	1136.32	N/A	N/A	N/A	N/A
2022.10	1263.58	N/A	N/A	N/A	N/A
2022.20	1397.28	N/A	N/A	N/A	N/A
2022.30	1537.07	N/A	N/A	N/A	N/A
2022.40	1682.67	N/A	N/A	N/A	N/A
2022.50	1833.82	N/A	N/A	N/A	N/A
2022.60	1990.29	N/A	N/A	N/A	N/A
2022.70	2151.90	N/A	N/A	N/A	N/A
2022.80	2318.48	N/A	N/A	N/A	N/A
2022.90	2489.85	N/A	N/A	N/A	N/A
2023.00	2665.89	N/A	N/A	N/A	N/A

Basin Water Elevation	Basin Outflow (cfs)	Riser Box Water Elevation	Tailwater Elevation (ft)	Outfall Culvert Control	Outfall Culvert Override?
2023.10	2846.46	N/A	N/A	N/A	N/A
2023.20	3031.44	N/A	N/A	N/A	N/A
2023.30	3220.73	N/A	N/A	N/A	N/A
2023.40	3414.21	N/A	N/A	N/A	N/A
2023.50	3611.80	N/A	N/A	N/A	N/A
2023.60	3813.41	N/A	N/A	N/A	N/A
2023.70	4018.95	N/A	N/A	N/A	N/A
2023.80	4228.35	N/A	N/A	N/A	N/A
2023.90	4441.54	N/A	N/A	N/A	N/A
2024.00	4658.43	N/A	N/A	N/A	N/A
2024.10	4878.98	N/A	N/A	N/A	N/A
2024.20	5103.11	N/A	N/A	N/A	N/A
2024.30	5330.77	N/A	N/A	N/A	N/A
2024.40	5561.91	N/A	N/A	N/A	N/A
2024.50	5796.46	N/A	N/A	N/A	N/A
2024.60	6034.38	N/A	N/A	N/A	N/A
2024.70	6275.63	N/A	N/A	N/A	N/A
2024.80	6520.14	N/A	N/A	N/A	N/A
2024.90	6767.88	N/A	N/A	N/A	N/A
2025.00	7018.81	N/A	N/A	N/A	N/A

C.3 TR-55 MODEL

Following are hydrographs produced for the 2-, 10-, 25-, 50-, and 100-year storms using the TR-55 tabular method in VTPSUHM. Inputs are shown only for the 2-year storm since the only variable between the various events is rainfall depth.

SCS TR55 Tabular Method

Watershed Title: Upper pond

2 Year Type II Storm: Precipitation = 3 inches

Summary of Input Parameters

Subarea	Area (sq. mi.)	Curve Number	IA/P	Runoff (in)	Tc (hours)	Adj. Tc (hours)	Tt (hours)	Adj. Tt (hours)
1	0.062	84	0.127	1.52	0.473	0.500	0.647	0.620
2	0.111	79	0.177	1.19	0.583	0.500	0.397	0.480
3	0.078	77	0.199	1.07	0.543	0.500	0.302	0.345
4	0.095	77	0.199	1.07	0.502	0.500	0.285	0.287
5	0.055	80	0.167	1.25	0.462	0.500	0.290	0.252
6	0.103	81	0.156	1.31	0.353	0.400	0.157	0.110
7	0.079	84	0.127	1.52	0.425	0.400	0.228	0.253
8	0.062	78	0.188	1.13	0.437	0.400	0.172	0.209
9	0.077	79	0.177	1.19	0.718	0.750	0.138	0.106
10	0.148	84	0.127	1.52	0.373	0.400	0.083	0.056
11	0.099	79	0.177	1.19	0.787	0.750	0.063	0.100
12	0.149	90	0.100	1.98	0.710	0.750	0.000	0.000
Composite	**1.118**	**82**		**1.37**				

SCS TR55 Tabular Method

Watershed Title: Upper pond

2 Year Type II Storm: Precipitation = 3 inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs)											
	11.0	**11.9**	**12.2**	**12.5**	**12.8**	**13.2**	**13.6**	**14.0**	**15.0**	**17.0**	**20.0**	**26.0**
1	0.8	1.9	3.1	10.4	27.1	29.9	16.9	8.8	3.9	2.4	1.6	0.0
2	0.9	2.3	4.0	18.9	47.7	36.3	17.3	10.2	5.7	3.6	2.4	0.0
3	0.5	1.4	3.4	18.9	32.6	18.2	9.0	5.9	3.6	2.3	1.6	0.0
4	0.7	1.8	5.4	28.6	39.0	19.5	10.0	6.8	4.3	2.8	1.9	0.0
5	0.6	1.7	5.2	23.9	26.1	11.8	6.1	4.2	2.7	1.8	1.2	0.0
6	1.7	6.3	38.4	63.9	30.6	13.1	8.8	7.0	4.9	3.2	2.2	0.0
7	1.6	4.4	17.3	57.1	39.8	15.1	8.4	6.3	4.2	2.7	1.8	0.0
8	0.6	1.7	8.6	32.6	21.9	8.7	5.2	4.0	2.7	1.8	1.2	0.0
9	0.7	1.9	5.4	23.3	33.0	19.0	10.4	6.8	3.9	2.4	1.7	0.0
10	3.5	13.9	87.1	101.6	42.3	19.1	13.3	10.7	7.5	5.0	3.4	0.0
11	0.9	2.5	7.2	30.6	42.5	24.2	13.2	8.7	5.0	3.1	2.1	0.0
12	3.8	10.6	34.0	112.3	109.1	50.8	27.5	18.0	10.3	6.5	4.4	0.0
Composite	**16.4**	**50.4**	**219.1**	**522.1**	**491.8**	**265.9**	**146.0**	**97.6**	**58.6**	**37.5**	**25.5**	**0.0**

The peak flow is 547.8 cfs at 12.6 hrs.

SCS TR55 Tabular Method

Watershed Title: Upper pond

10 Year Type II Storm: Precipitation = 4.9 inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs)											
	11.0	**11.9**	**12.2**	**12.5**	**12.8**	**13.2**	**13.6**	**14.0**	**15.0**	**17.0**	**20.0**	**26.0**
1	2.0	4.7	7.5	24.5	59.4	63.2	35.3	18.0	7.9	4.7	3.1	0.0
2	3.2	8.1	14.1	58.9	123.7	80.3	35.6	20.6	11.3	7.0	4.6	0.0
3	2.2	5.7	13.7	58.5	81.2	40.9	19.2	12.3	7.3	4.6	3.1	0.0
4	2.8	7.6	21.5	88.1	95.3	41.8	20.7	14.0	8.6	5.6	3.8	0.0
5	2.1	5.6	16.2	62.5	60.1	24.8	12.4	8.5	5.3	3.4	2.3	0.0
6	5.3	19.3	108.7	142.6	61.9	25.9	17.2	13.8	9.6	6.3	4.2	0.0
7	3.8	10.7	40.7	125.4	82.9	30.4	16.9	12.6	8.3	5.3	3.5	0.0
8	2.3	6.6	27.2	83.1	50.9	18.8	10.9	8.2	5.5	3.6	2.4	0.0
9	2.6	6.7	18.9	67.2	79.3	41.0	21.5	13.9	7.7	4.7	3.2	0.0
10	8.5	33.6	198.3	213.9	85.4	38.1	26.4	21.2	14.8	9.9	6.6	0.0
11	3.3	8.8	25.0	88.3	101.8	51.8	27.2	17.8	9.9	6.0	4.1	0.0
12	7.3	20.3	64.8	214.0	207.8	96.9	52.4	34.4	19.7	12.4	8.4	0.0
Composite	**45.3**	**137.6**	**556.7**	**1226.9**	**1089.9**	**553.9**	**295.7**	**195.3**	**115.8**	**73.5**	**49.3**	**0.0**

The peak flow is 1262.8 cfs at 12.6 hrs.

SCS TR55 Tabular Method

Watershed Title: Upper pond

25 Year Type II Storm: Precipitation = 5.4 inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs)											
	11.0	**11.9**	**12.2**	**12.5**	**12.8**	**13.2**	**13.6**	**14.0**	**15.0**	**17.0**	**20.0**	**26.0**
1	2.3	5.3	8.6	28.1	68.0	72.3	40.4	20.6	9.0	5.4	3.5	0.0
2	3.9	9.9	17.1	70.5	145.5	92.8	40.8	23.5	12.9	8.0	5.2	0.0
3	2.7	7.1	16.9	70.5	95.4	47.3	22.0	14.0	8.3	5.2	3.6	0.0
4	3.5	9.4	26.6	106.1	111.5	48.0	23.6	16.0	9.8	6.4	4.3	0.0
5	2.4	6.5	18.9	72.6	69.6	28.6	14.3	9.7	6.1	3.9	2.6	0.0
6	6.1	22.3	125.4	164.5	71.3	29.9	19.9	15.9	11.0	7.2	4.8	0.0
7	4.3	12.2	46.6	143.5	94.9	34.8	19.3	14.4	9.5	6.0	4.0	0.0
8	2.8	8.1	32.8	97.8	59.2	21.6	12.5	9.4	6.3	4.0	2.7	0.0
9	3.1	8.2	22.9	79.9	92.5	47.2	24.6	15.9	8.8	5.3	3.6	0.0
10	9.7	38.4	227.0	244.7	97.7	43.6	30.2	24.3	17.0	11.3	7.5	0.0
11	4.1	10.6	30.2	105.1	118.8	59.5	31.2	20.3	11.2	6.9	4.7	0.0
12	8.3	22.9	73.0	241.4	234.4	109.3	59.1	38.7	22.2	14.0	9.5	0.0
Composite	**53.1**	**160.8**	**646.1**	**1424.7**	**1259.0**	**634.9**	**337.8**	**222.8**	**132.0**	**83.6**	**56.1**	**0.0**

The peak flow is 1465.0 cfs at 12.6hrs.

SCS TR55 Tabular Method

Watershed Title: Upper pond

50 Year Type II Storm: Precipitation = 6 inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs)											
	11.0	**11.9**	**12.2**	**12.5**	**12.8**	**13.2**	**13.6**	**14.0**	**15.0**	**17.0**	**20.0**	**26.0**
1	2.6	6.2	9.9	32.4	78.5	83.4	46.6	23.8	10.4	6.2	4.0	0.0
2	4.6	11.5	20.0	82.4	170.1	108.4	47.6	27.4	15.0	9.3	6.1	0.0
3	3.4	8.8	20.9	85.5	112.8	55.2	25.5	16.2	9.5	6.0	4.1	0.0
4	4.3	11.7	32.9	128.5	131.5	55.6	27.2	18.4	11.2	7.3	4.9	0.0
5	2.8	7.6	22.1	84.6	81.1	33.4	16.7	11.3	7.1	4.6	3.0	0.0
6	7.1	25.9	145.8	191.2	82.9	34.7	23.1	18.5	12.8	8.4	5.6	0.0
7	5.0	14.1	53.7	165.5	109.5	40.1	22.3	16.6	10.9	7.0	4.6	0.0
8	3.3	9.7	39.1	115.2	69.4	25.2	14.5	10.9	7.3	4.7	3.1	0.0
9	3.6	9.5	26.7	93.3	108.1	55.2	28.8	18.6	10.2	6.3	4.2	0.0
10	11.2	44.3	261.8	282.3	112.7	50.3	34.9	28.0	19.6	13.0	8.7	0.0
11	4.7	12.4	35.3	122.8	138.8	69.6	36.4	23.7	13.1	8.0	5.5	0.0
12	9.4	26.0	83.0	274.4	266.4	124.2	67.1	44.0	25.3	15.9	10.8	0.0
Composite	**62.1**	**187.6**	**751.2**	**1658.0**	**1461.9**	**735.2**	**390.6**	**257.5**	**152.5**	**96.6**	**64.7**	**0.0**

The peak flow is 1704.2 cfs at 12.6 hrs.

SCS TR55 Tabular Method

Watershed Title: Upper pond

100 Year Type II Storm: Precipitation = 6.5 inches

Individual Subarea and Composite Hydrographs

Subarea	Time (hrs)											
	11.0	**11.9**	**12.2**	**12.5**	**12.8**	**13.2**	**13.6**	**14.0**	**15.0**	**17.0**	**20.0**	**26.0**
1	2.9	6.9	11.0	36.0	87.3	92.7	51.8	26.5	11.5	6.9	4.5	0.0
2	5.1	12.9	22.5	92.5	190.9	121.6	53.4	30.8	16.9	10.5	6.9	0.0
3	3.8	9.9	23.6	96.3	127.1	62.2	28.7	18.2	10.7	6.7	4.6	0.0
4	4.9	13.2	37.0	144.7	148.2	62.6	30.6	20.7	12.7	8.2	5.5	0.0
5	3.1	8.5	24.7	94.8	90.8	37.4	18.7	12.7	7.9	5.1	3.4	0.0
6	7.9	28.9	163.0	213.8	92.7	38.8	25.8	20.7	14.4	9.4	6.3	0.0
7	5.5	15.6	59.7	184.0	121.7	44.6	24.7	18.5	12.2	7.7	5.2	0.0
8	3.7	10.9	43.9	129.5	78.0	28.3	16.3	12.3	8.2	5.2	3.5	0.0
9	4.1	10.7	30.0	104.7	121.3	61.9	32.3	20.9	11.5	7.0	4.8	0.0
10	12.4	49.3	291.1	313.9	125.3	55.9	38.8	31.2	21.8	14.5	9.7	0.0
11	5.3	13.9	39.7	137.8	155.8	78.1	40.9	26.6	14.7	9.0	6.1	0.0
12	10.3	28.6	91.4	302.0	293.3	136.7	73.9	48.5	27.8	17.5	11.9	0.0
Composite	**69.3**	**209.3**	**837.6**	**1850.0**	**1632.4**	**820.9**	**435.9**	**287.4**	**170.2**	**107.8**	**72.3**	**0.0**

The peak flow is 1902.1 cfs at 12.6 hrs.

APPENDIX D

DATA FOR WEIR #2

Table D.1 – Experimental data from weir #2.

WSEL	Pump 1				Pump 2			
	L	R	DH	Q	L	R	DH	Q
cm	ft	ft	ft	cfs	ft	ft	ft	cfs
26.97	1.680	1.690	0.010	0.0057				
27.32	1.670	1.710	0.040	0.0114				
27.45	1.665	1.720	0.055	0.0134				
27.71	1.640	1.755	0.115	0.0193				
27.84	1.630	1.780	0.150	0.0221				
27.95	1.610	1.815	0.205	0.0258				
28.03	1.605	1.845	0.240	0.0279				
28.13	1.580	1.885	0.305	0.0315				
28.23	1.560	1.925	0.365	0.0344				
28.41	1.500	2.005	0.505	0.0405				
28.51	1.465	2.070	0.605	0.0443				
Data above are for small orifice; data below are for large orifice.								
27.90	1.810	1.820	0.010	0.0201				
28.26	1.815	1.830	0.015	0.0246				
28.56	1.815	1.840	0.025	0.0318				
28.85	1.810	1.855	0.045	0.0426				
29.12	1.805	1.870	0.065	0.0512				
29.33	1.795	1.885	0.090	0.0603				
29.55	1.780	1.900	0.120	0.0696				
29.71	1.770	1.920	0.150	0.0778				

Table D.1 (continued) – Experimental data for weir #2.

WSEL	Pump 1				Pump 2			
	L	R	DH	Q	L	R	DH	Q
cm	ft	ft	ft	cfs	ft	ft	ft	cfs
29.84	1.755	1.935	0.180	0.0852				
30.00	1.730	1.950	0.220	0.0942				
30.15	1.710	1.970	0.260	0.1024				
30.31	1.675	1.985	0.310	0.1119				
30.49	1.625	2.005	0.380	0.1238				
30.62	1.585	2.025	0.440	0.1333				
30.73	1.540	2.040	0.500	0.1421				
30.81	1.505	2.050	0.545	0.1483				
30.93	1.450	2.065	0.615	0.1575				
31.05	1.390	2.080	0.690	0.1669				
31.13	1.350	2.085	0.735	0.1722				
31.22	1.290	2.090	0.800	0.1797				
31.45	1.290	2.090	0.800	0.1797	1.165	1.155	0.010	0.0201
31.52	1.290	2.090	0.800	0.1797	1.170	1.155	0.015	0.0246
31.62	1.290	2.090	0.800	0.1797	1.175	1.150	0.025	0.0318

Table D.1 contains the experimental data taken from the model for weir #2. Table D.2 shows the transformed data using the length and flow ratios (L_r *and* Q_r) given below:

$$L_r = \frac{L_m}{L_p} = \frac{12''}{225''} = 0.05\overline{3} \qquad \text{[D-1]}$$

$$Q_r = L_r^{2.5} = \left(0.05\overline{3}\right)^{2.5} = 6.569\times10^{-4} \qquad \text{[D-2]}$$

Table D.2 – Rating curve data for prototype weir #2.

Model		Prototype		
Depth *ft*	**Flow** *cfs*	**Depth** *ft*	**Flow** *cfs*	
0.054	0.006	1.01	8.7	Pump #1
0.065	0.011	1.22	17.4	(small orifice)
0.070	0.013	1.30	20.3	
0.078	0.019	1.46	29.4	
0.082	0.022	1.54	33.6	
0.086	0.026	1.61	39.3	
0.089	0.028	1.66	42.5	
0.092	0.031	1.72	47.9	
0.095	0.034	1.78	52.4	
0.101	0.041	1.89	61.7	
0.104	0.044	1.96	67.5	
0.084	0.020	1.58	30.6	Pump #1
0.096	0.025	1.80	37.5	(large orifice)
0.106	0.032	1.99	48.4	
0.115	0.043	2.17	64.9	
0.124	0.051	2.33	78.0	
0.131	0.060	2.46	97.2	
0.138	0.070	2.60	111.4	
0.144	0.078	2.69	123.9	

Table D.2 (continued) – Rating curve data for prototype weir #2.

Model		Prototype		
Depth *ft*	**Flow** *cfs*	**Depth** *ft*	**Flow** *cfs*	
0.148	0.085	2.77	135.3	Pump #1
0.153	0.094	2.87	148.9	(large orifice)
0.158	0.102	2.97	161.4	
0.163	0.112	3.06	175.8	
0.169	0.124	3.17	194.0	
0.174	0.133	3.25	208.4	
0.177	0.142	3.32	221.8	
0.180	0.148	3.37	231.3	
0.184	0.158	3.44	245.3	
0.188	0.167	3.52	259.5	
0.190	0.172	3.57	267.7	
0.193	0.180	3.62	279.0	
0.201	0.200	3.76	309.6	Pump #1
0.203	0.204	3.81	316.5	and pump #2
0.206	0.211	3.87	327.4	(large orifice)

Figure D.1 shows the above data graphically. The curves for the small and large orifice data do not match well, so the overlapping points were eliminated from the data set to create figure D.2. The equation for the best-fit line in figure D.2 is a power function, and the exponent is near 2.5 as would be expected for a triangular weir.

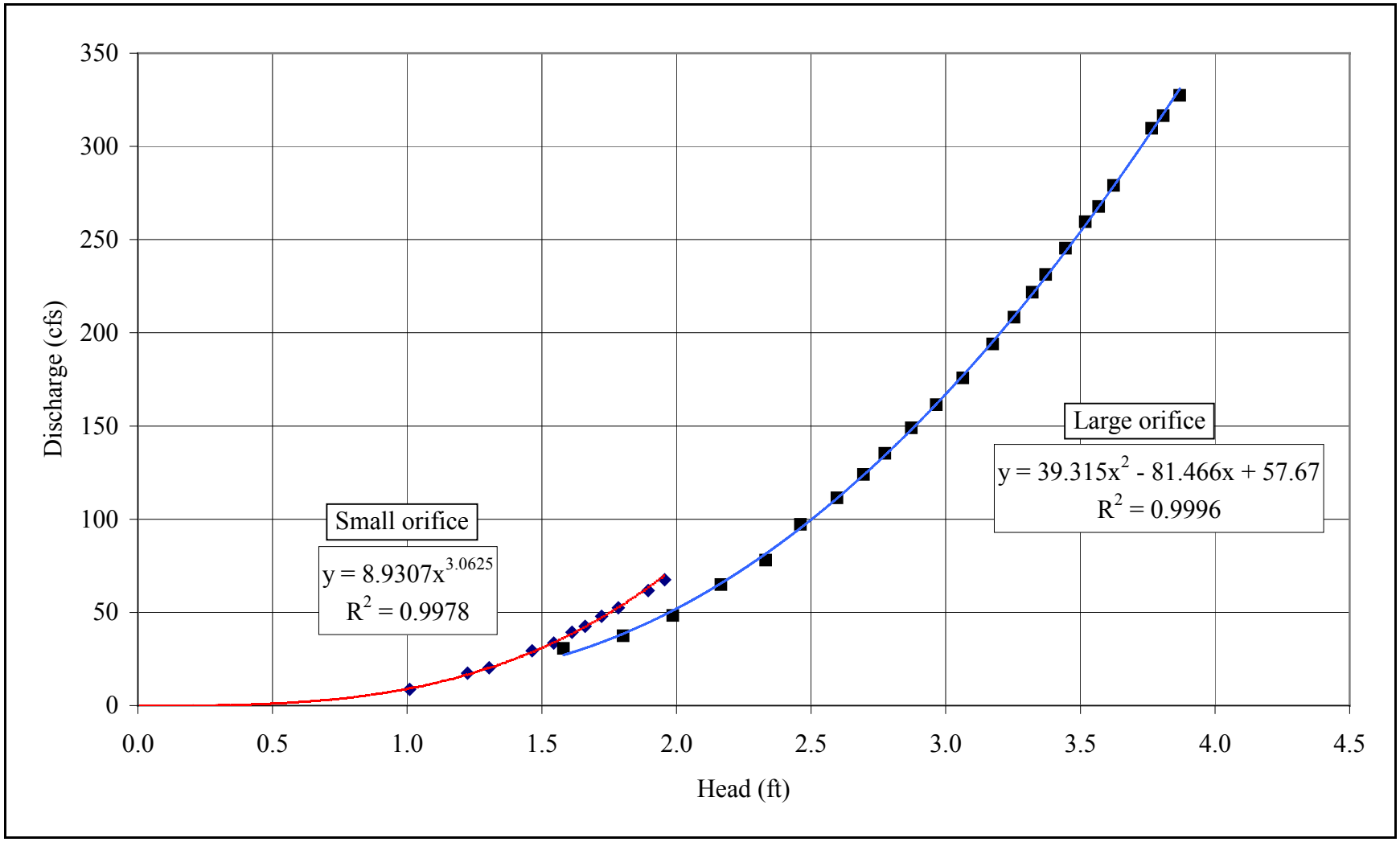

Figure D.1 – Unadjusted depth-discharge rating curve for prototype weir #2.

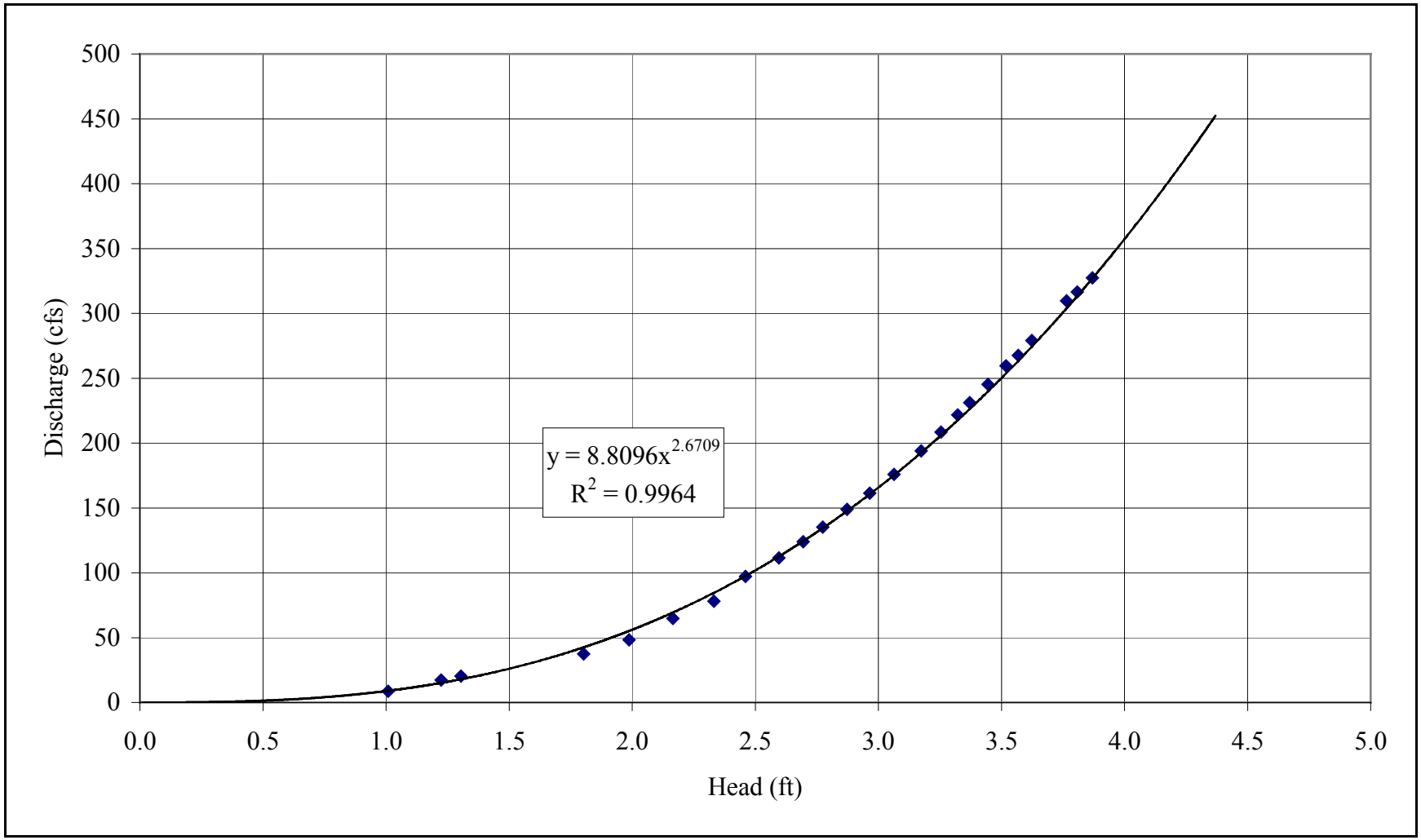

Figure D.2 – Depth-discharge rating curve for weir #2; overlapping points between large and small orifice deleted from data set.

APPENDIX E

FINAL CONSTRUCTION DOCUMENTS

The following are links to construction plans for this project. These plans were created in AutoCAD 2000i and printed to 24”x36” (full-size) using Acrobat Distiller. The Virginia Tech Capital Design & Construction department will begin construction in the spring of 2003 and should complete this phase of the project by the fall of 2004.

Sheet #	Title
1	Title Sheet
2	Dam Modifications
3	S. Branch Stabilization
4	Profile Sheet
5	Weir Construction Details
6	Site Details

APPENDIX F

EPA-SWMM INPUT AND OUTPUT FILES

The following are links to the data files used in EPA-SWMM version 4.4H. These files are grouped together in a folder titled "EPA-SWMM data files" and can be viewed using any text editor. Input files have a ".txt" extension, output files have a ".out" extension. The program and user files for EPA-SWMM version 4.4H may be found on the University of Oregon's website <http://www.ccee.orst.edu/swmm/>.

UNADJUSTED MODEL

EXTRAN block input
2-year runoff input
2-year runoff output
2-year EXTRAN output
10-year runoff input
10-year runoff output
10-year EXTRAN output
25-year runoff input
25-year runoff output
25-year EXTRAN output
50-year runoff input
50-year runoff output
50-year EXTRAN output
100-year runoff input
100-year runoff output
100-year EXTRAN output

25-YEAR ADJUSTED MODEL

EXTRAN block input
2-year EXTRAN output
10-year EXTRAN output
25-year EXTRAN output
50-year EXTRAN output
100-year EXTRAN output

100-YEAR ADJUSTED MODEL

EXTRAN block input
2-year EXTRAN output
10-year EXTRAN output
25-year EXTRAN output
50-year EXTRAN output
100-year EXTRAN output

VITA

Forrest Brian Thye, son of Forrest Wallace Thye and Lynn Rockcastle Thye, was born on November 4th, 1973 in Blacksburg, Virginia. He hasn't yet found a reason to leave. Brian attended Blacksburg High School, took two years of drafting courses, and graduated in 1991. After a "false start" at Virginia Tech in the '91-'92 academic year, he went to work full-time as a drafter at Anderson & Associates, Inc., a local consulting civil engineering firm, where he had been hired as a high-school senior. He graduated from New River Community College in 1995 with an A.A.S. in Drafting and Design. Brian re-entered the Virginia Tech College of Engineering in the summer of 1998 after spending two years at Virginia Western Community College, and graduated Summa cum Laude with a B.S. in Civil Engineering in the spring of 2000. He spent three years at Tech as a Pratt scholar, and taught Water Resources Engineering labs for three semesters. This project report will complete his M.S. in Civil Engineering. Brian continues to work at Anderson & Associates, Inc. as an Assistant Project Manager in Land Development and plans to take the Virginia Principles and Practice of Engineering exam in the fall of 2003.